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THE Association was organized in Boston, Mass., on June 21, 1882, with the object of providing its members with means of social intercourse and for the exchange of knowledge pertaining to the construction and management of water works. From an original membership of only TWENTY-SEVEN, its growth has prospered until now it includes the names of 900 men. Its membership is divided into six classes, viz.:

A Member shall be an officer or employee of a public or private water works, an engineer, chemist or other person qualified to aid or interested in the advancement of knowledge relative to water works.

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A Corporate Member shall be either a Water Board, Commission, Company or Municipal Corporation.

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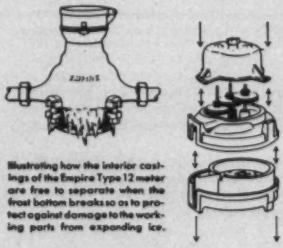

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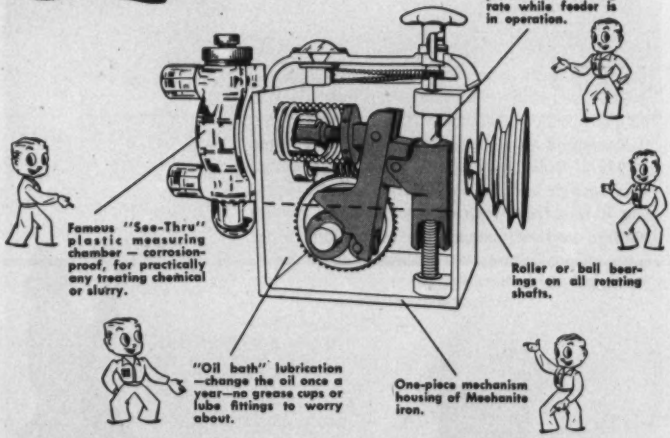
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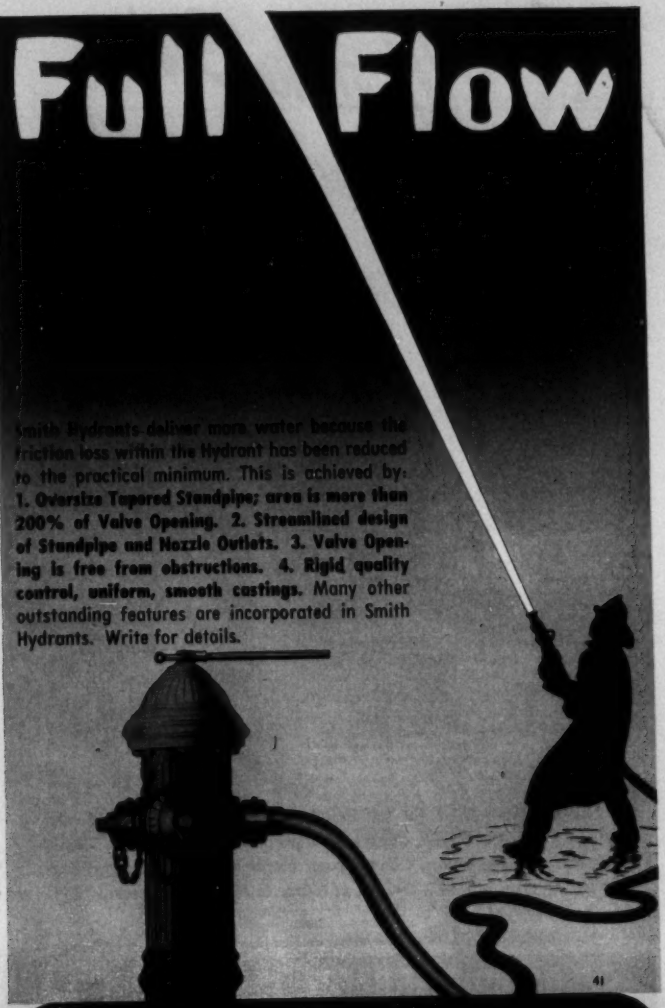

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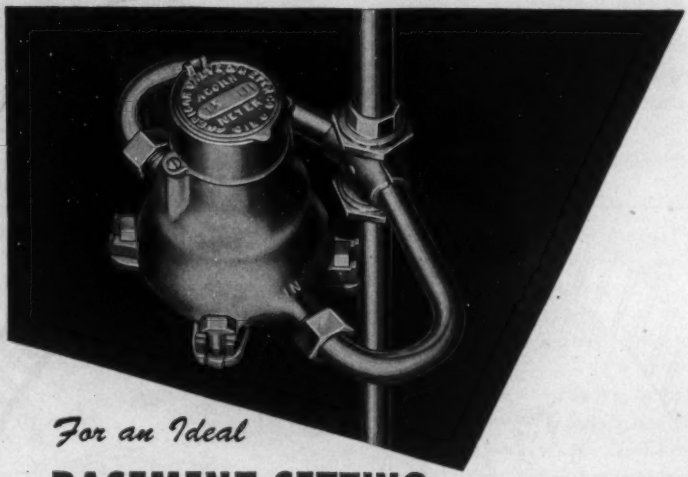

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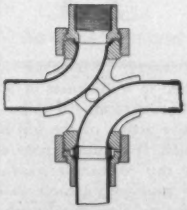

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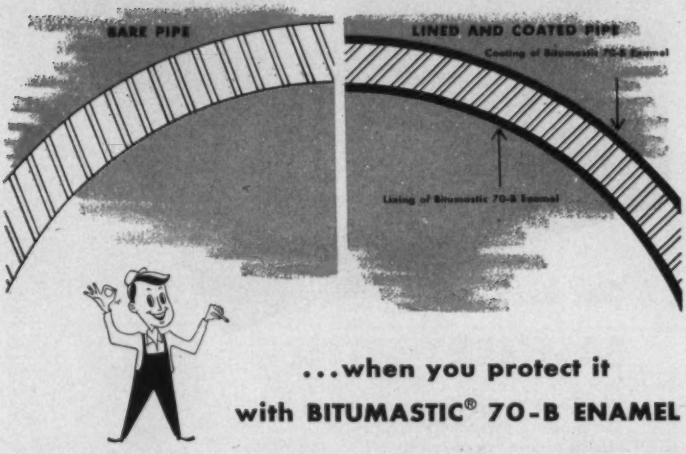

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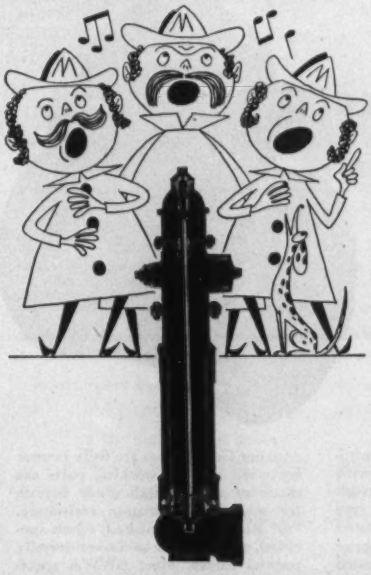

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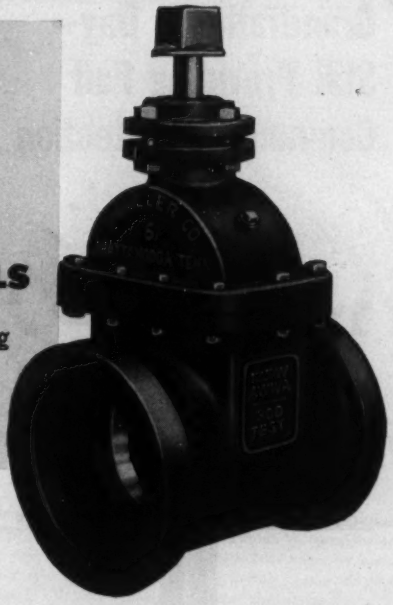

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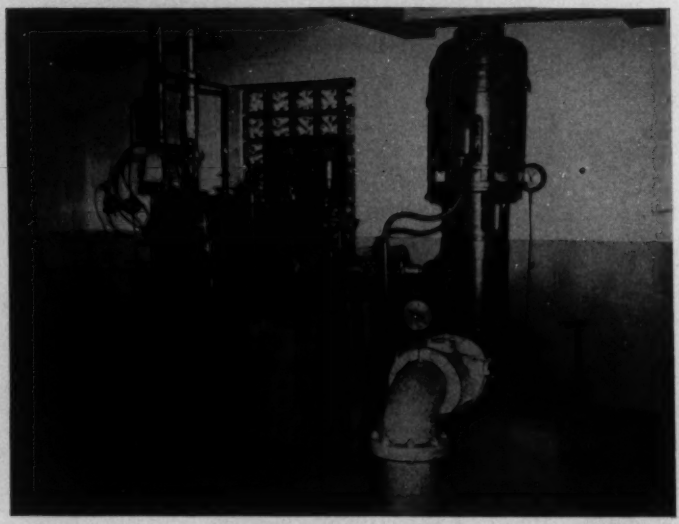

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New England Water Works Association
ORGANIZED 1882

VOL. LXVII JUNE, 1953 No. 2

This Association, as a body, is not responsible for the statements or opinions of any of its members

GEOLOGIC FACTORS AFFECTING THE YIELD OF ROCK WELLS IN SOUTHERN NEW ENGLAND*

BY R. V. CUSHMAN, W. B. ALLEN AND H. L. PREE, JR.†

[*Read February 15, 1951.*]

INTRODUCTION

In its ground-water investigations in coöperation with the Connecticut State Water Commission, the Massachusetts Department of Public Works and the Rhode Island Development Council, the United States Geological Survey receives a variety of inquiries pertaining to the procurement of a water supply from bedrock for rural homes, industrial plants and municipalities. "Where shall I drill?", "How deep must I drill?", and "How much water will I obtain?" are the questions most frequently asked. Those of you who have faced similar problems no doubt realize that in New England the answer is likely to be different for each request. The factors that determine the occurrence of ground water are not as apparent in New England as in regions where geologic structures remain unchanged over large areas, and where the bedrock is not mantled by an extensive cover of overburden.

Lacking occult powers and "X-ray eyes," as well as sufficient data of a scientific nature regarding the physical and hydraulic characteristics of bedrock at most localities, we can provide only general answers to these requests. The possibilities of obtaining water supplies may be expressed as averages, computed from data collected from drillers and owners of rock wells in New England. Statistical analysis of the well records is helpful in understanding the results obtained by drilling in rock. The purpose of this paper is to show how certain geologic factors affect the yield of wells in bedrock and

*Published with the approval of the Director, U. S. Geological Survey.
†Geologists, U. S. Geological Survey, Ground Water Branch.

how these effects may be recorded statistically. Basic data are obtained from a correlative study of the records of about 2,000 rock wells, locally called artesian wells in southern New England. Only those wells that end in and obtain water from the solid rock or bedrock are included. All are cased at least as far as the bedrock surface. Many of the wells considered have been drilled since about 1930. The records for most wells are accurate, but are limited in scope and provide only a minimum of basic data.

The Geological Survey is making ground-water investigations in coöperation with the States of Connecticut, Massachusetts and Rhode Island, and maintains offices at Middletown, Boston and Providence, in those respective States. The studies are financed by equal appropriations from the State and Federal governments. They are under the supervision of M. L. Brashears, Jr., District Geologist for the Ground Water Branch of the Geological Survey in New York and New England. This paper is a coöperative project of the three offices of the Geological Survey in New England and is based on data obtained from their open files. Additional information was furnished by the three coöperating State agencies and by other State agencies. The assistance of well drillers and property owners, without which this report could not have been written, is gratefully acknowledged.

Occurrence of Ground Water in Rocks

Ground water, or well water, is that part of the subsurface water that occurs in the zone of saturation. It is derived essentially from rain and snow melt that percolates into the soil and moves downward to the subsurface reservoir. It is estimated that an average of about one-sixth of the precipitation in New England reaches the zone of saturation and is available for withdrawal from wells.

Ground water moves under the influence of gravity and is always discharged at a lower level than the area of intake or replenishment. In New England replenishment generally occurs in interstream areas and the natural discharge is to streams, lakes and other bodies of surface water. The upper surface of the zone of saturation is called the water table. In New England the water table is a few feet to as much as 100 feet below the land surface, and

rises and falls from season to season and year to year in accordance with variations in precipitation, surface runoff, withdrawals by wells, and other related factors.

Within the saturated zone, ground water is in pore spaces or other openings in rocks. The water-bearing properties of rocks are determined largely by the nature and size of these openings. In granular deposits, such as the unconsolidated glacial drift of New England shown in Plate 1, water is contained in primary openings

PLATE 1.—PLEISTOCENE OUTWASH AT PLAINVILLE, MASSACHUSETTS.
(Photograph by Alonzo W. Quinn, Brown Univ., Providence, R. I.)

or pore spaces, which may constitute as much as 50% of the volume of the drift. Thus, beds of coarse-textured drift are highly per-

meable and usually yield large quantities of water. In contrast, most of the solid rocks of New England contain less than 1% of pore space and have low permeability, which limits the movement of ground water. Nearly all the water in bedrock is contained in secondary openings such as fractures.

Bedrock of Southern New England

The bedrock of southern New England consists of igneous, metamorphic and sedimentary rocks, that may be divided into a number of recognized geologic formations. However, because nearly all the consolidated rocks in New England have a porosity of less than 1% (though sandstone of Triassic age has an average porosity of about 7%), a detailed sub-division of the geologic formations is of little significance in discussing the occurrence and movement of ground water. For the purposes of this study the consolidated rocks of southern New England have been divided into the five groups shown below (Fig. 1):

1. Pre-Triassic crystalline rocks;
2. Paleozoic limestone;
3. Carboniferous sedimentary rocks;
4. Triassic sedimentary rocks;
5. Triassic igneous rocks.

The rocks of pre-Triassic age are largely metamorphosed and have been greatly altered from their original sedimentary or igneous character. The metamorphic rocks are complexly intermingled with igneous rocks, the group consisting chiefly of granite, granite gneiss, gneiss, and schist. They are generally known as crystalline rocks and are widespread throughout southern New England, as is shown on the accompanying map (Fig. 1).

The limestone of Paleozoic age is chiefly dolomite marble. These rocks are, in effect, crystalline rocks, as they have been recrystallized from original sediments. However, they are considered separately here, because they consist in large part of comparatively soluble minerals and generally have larger and more numerous secondary openings. They form a discontinuous belt in western Connecticut and Massachusetts.

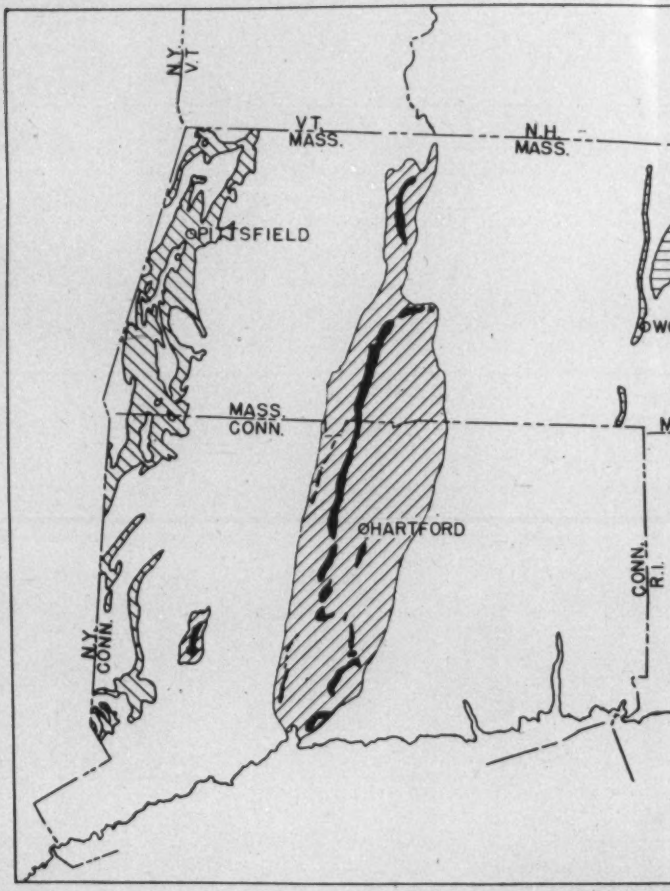
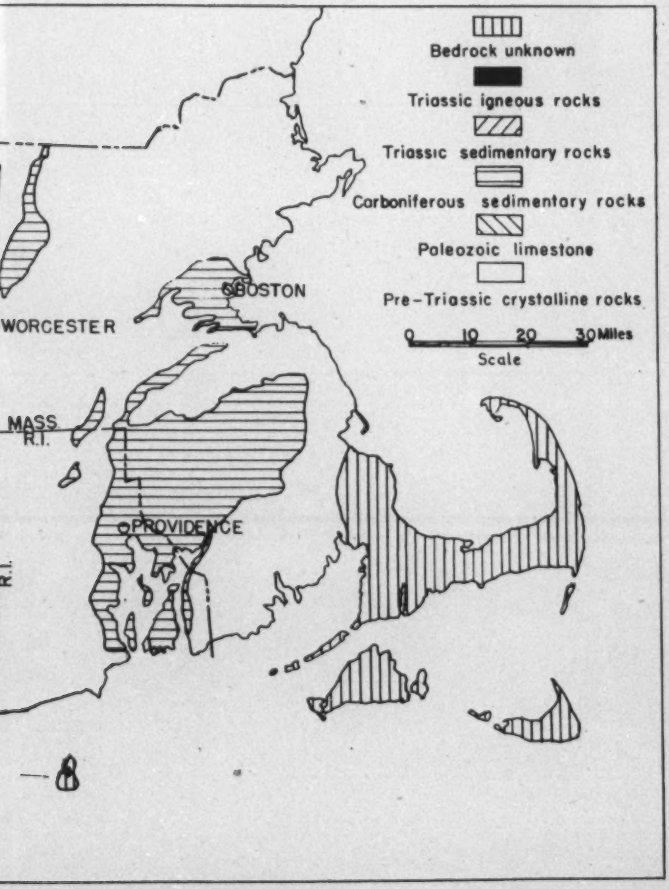


FIG. 1.—GENERALIZED GEOLOGIC M



MAP OF SOUTHERN NEW ENGLAND.

Included in the third group are the Carboniferous rocks of the Boston, Worcester, Woonsocket and Narragansett basins. They consist of tightly folded and metamorphosed conglomerate, sandstone, arkose, shale, and slate. In general they have undergone a lesser degree of metamorphism than the other pre-Triassic rocks. They occupy troughs or basins bounded in part by depositional or fault contacts with older crystalline rocks and in part by intrusive contacts with younger igneous rocks.

The youngest consolidated rocks in southern New England are the Triassic sedimentary and igneous rocks. The Triassic sediments are the distinctive red sandstone, shale, and conglomerate that underlie most of the broad Connecticut River lowland in Connecticut and Massachusetts. They constitute a series of parallel strata, separated by more or less definite bedding planes, and are interbedded with several thick layers or flows of dense basalt or "trap" rock. The "trap" rock is more resistant to erosion than the red sediments and generally stands up as ridges in the otherwise nearly flat valley floor.

Conditions Affecting the Accuracy of Results

Although this study of geologic factors affecting the yield of rock wells was made with an attempt at obtaining maximum accuracy, it is realized that certain conditions exist that affect the accuracy of the results obtained. The first of these conditions concerns the reliability of the records used. The writers selected wells for which reasonably accurate records are available. Many wells were eliminated from the study because complete records were not available. However, because some of the data were obtained directly from written records of well drillers and owners, and others were based on the memory of drillers and owners, the degree of accuracy may vary with the source.

A second condition that influences the accuracy of the results to a great degree is the difficulty of isolating the effect of a single factor on the yield of a rock well. Material was assembled in each phase of the study to indicate the effect of some particular factor on yield, as, for instance, the type of overburden. Yet all the other factors, such as topography and structure, also control the yield to some extent and are difficult to remove from the calculation. More

accurate results might have been obtained if wells could have been chosen for each table so as to eliminate all but one effect. However, owing to limitations of time and accuracy of well records, it was impossible to do this.

Thirdly, the reported yield of a well is dependent upon the accuracy and completeness of the pumping test used to determine that yield. The yield of the majority of wells probably was determined by a bailing test, wherein the driller bailed water out of the well in a standard bailer and found that he could remove—for example—5 gallons of water per minute. Unfortunately there are no prescribed rules for testing the yield of a well, and drillers vary the rate and duration of bailing or pumping from well to well. It is probable that the reported yield was enough to satisfy the needs of the well owner, but that in nearly all instances it represented only a fraction of the possible yield of the well, if a more exhaustive test were conducted, or if the well were more fully developed. Furthermore, a large number of the wells included in the study are wells drilled to supply a moderate amount of water for rural homes. When the desired yield was obtained, drilling was stopped. In contrast, a much smaller number of wells were drilled for industrial plants requiring larger quantities of water, and drilling was therefore continued until either the desired yield was furnished or the funds allotted for drilling were exhausted. It is probable that figures for the average yield of wells obtained in the study are too small, and that most rocks could yield more water if the wells were drilled deeper and developed; therefore, the figure for average depth of wells in a given type of rock, based on all the data, is lower than the average depth necessary to drill for maximum yield. This is clearly shown in the following table, which compares the yield of all wells in Triassic sediments with such data for 44 industrial wells tapping these rocks in Hartford, Conn.

Unit	Number of wells	Average depth (feet)	Average yield (gpm)
All wells in Triassic sediments	553	199	19.8
Industrial wells in Triassic sediments in Hartford, Conn.	44	513	131.8

Factors Affecting the Yield of Rock Wells

The amount of water that may be pumped continuously from a well in bedrock is dependent upon the quantity of water entering or recharging the rock and the rate at which it will move through the rock to the well. The amount of precipitation that enters the rock is controlled in part by the topography and surface vegetation, which affect the amount of surface runoff. The amount and the rate of entry are determined also by the infiltration capacity of the overburden or soil material. The type of overburden and its thickness also control its transmitting and storage capacity. Finally, the rate at which water will move into and through the bedrock to the well is controlled by the type and structure of the rock itself. The larger and more numerous the fractures and the greater the extent to which they are interconnected, the greater the yield.

The effect on the yield of rock wells of these several factors—namely, rock type, structure, topography, and depth and type of overburden—are therefore discussed in detail.

Rock Type

Nearly all the consolidated rocks of southern New England are dense and compact and have a porosity of less than 1% (Triassic sandstone has an average porosity of about 7%). Thus, the primary physical characteristics of the various rocks have little effect on the relative yield of wells. However, because of characteristics associated with internal structure the various rocks react differently to geologic forces imposed on them subsequent to their deposition and consolidation. Secondary openings capable of transmitting and storing water are thus developed to a greater extent in some rocks than in others. For instance, fracturing of the coarser-grained and more brittle rocks, such as granite and trap rock, generally produces wider and more continuous joint openings than fracturing in finer-grained rocks. In contrast, planes of cleavage are better developed and more closely spaced in the finer-grained rocks. Thus, rock type has an important bearing on the development of secondary openings and consequently on the yield of rock wells.

Structure

Almost all the rocks have some structural planes along which openings capable of transmitting water may be developed. In southern New England the most prominent secondary structures are joints. The most common type of joint is that which dips at an angle greater than 45° from the surface, here called a vertical joint. These joints intersect each other at various angles and break the rock into irregular polygons. The width of opening varies with rock type and ranges from that which is scarcely visible to several inches. The spacing of vertical joints is usually 5 to 10 feet, but may range from a fraction of an inch to as much as 200 feet. The width of vertical joints and the number of joints decrease with depth, owing to increased pressure and the resulting decreased opportunity for lateral expansion. Thus, there is a greater circulation of water in the upper part of a given joint than in the lower part. This is an important consideration in the drilling of wells in consolidated rocks, as it is often thought that more water can always be obtained by deeper drilling. Actually, the deeper a well is drilled below a certain depth the less is the chance of striking fractures.

A second type of joint is horizontal or nearly horizontal. Such joints are due to the process known as sheeting and are well developed in the more massive rocks, particularly granite. The joint surfaces are more or less conformable to the ground level, being convex upward on the hills and concave upward in the valleys. Those in the valleys thus form shallow basins which are ideal for the collection and storage of ground water. The spacing of horizontal joints is more regular than that of vertical joints but their number and width diminish rapidly with depth. It is probable that they do not exist at depths much below 300 feet. Above this depth there is a widespread intersection of vertical and horizontal joints, affording opportunity for the lateral transmission of water from one joint to another. Plates 2, 3 and 4 illustrate the types of joints common to New England rocks.

Openings along other structural planes, such as those of schistosity or cleavage, are of secondary importance. They are well developed in the softer rocks like schist and separate the rock into irregular thin layers. They dip at all angles, but usually at greater angles than in the major sets of joints.

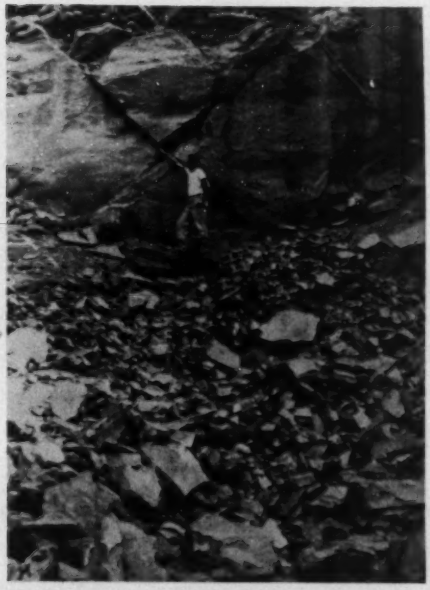


PLATE 2.—RHODE ISLAND FORMATION AT PLAINVILLE, MASS., SHOWING VERTICAL JOINTS. (Photograph by Alonzo W. Quinn, Brown Univ., Providence, R. I.)

Because circulation of ground water in the bedrock of southern New England is confined mainly to fractures, a comparison of the yield of wells in the various rock types may be used as a measure of the abundance of these fractures and their size and degree of interconnection. This comparison is made in Table 1. The table shows the average depth and yield of wells in the various rock units in southern New England. The major rock units are arranged in order of decreasing average yield. This order is followed in Tables 2 and 3 also. The wells included in Table 1 are in rocks overlain by all types and thicknesses of overburden in all topographic locations.

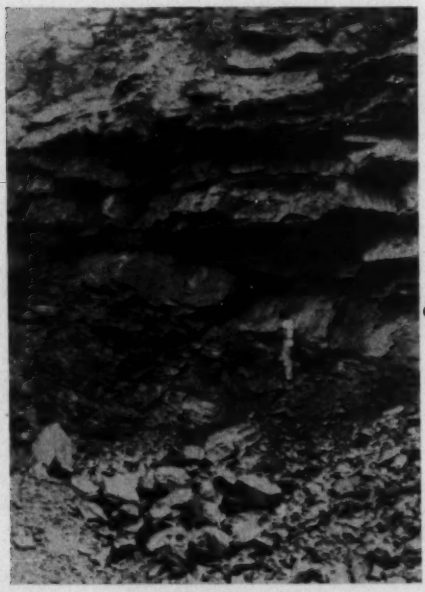


PLATE 3.—RHODE ISLAND FORMATION AT PLAINVILLE, MASS., SHOWING BEDDING AND JOINTING.

(Photograph by Alonzo W. Quinn, Brown Univ., Providence, R. I.)

Thus, it is hoped that the effects of these factors on yield have been equalized and that the relations shown in Table 1 are due only to the effects of geologic structure and rock type. The data show that the belt of metamorphosed limestone in the western part of the area has the highest average yield of the water-bearing units for which statistical data are available. Though the limestone is usually more irregularly fractured than the more massive rocks, the fracturing is closely spaced and interconnecting, thus offering easy passage for circulating ground water. Many fractures have, in addition, been enlarged by

PLATE 4.—TRIASSIC SANDSTONE AT PORTLAND, CONN., SHOWING SEEPAGE OF WATER (DARK AREAS) FROM JOINT AND BEDDING PLANES.

solution, as the calcium carbonate constituting the bulk of the rock is readily soluble in water containing carbon dioxide. The combination of shattering and solution has produced more open space than in any other type of rock and is reflected in the high average yield. An altered phase of the limestone encountered in several localities consists of a coarse-textured cellular rock, that is reported to yield large quantities of water to wells.

It should be pointed out, however, that the large figure for the average yield of wells in limestone, 177 gpm, is misleading not only because of the effect of the well of largest yield, as noted in Table 1, but as an indication of the comparison between limestone and other rocks. About one-third of the wells included in the average for limestone are deep wells, drilled for industrial purposes demanding larger quantities of water than are needed for most domestic wells. Several of these wells are known to penetrate the cellular zone of the limestone. In contrast, the number of industrial wells included in the average for the other rock units is generally less than 5%. The

TABLE 1.—SUMMARY OF DEPTHS AND YIELDS OF WELLS ENDING IN THE SEVERAL ROCK UNITS

Rock unit	No. of wells	Range in depth (feet)	Average depth (feet)	Range in yield (gpm)	Average yield (gpm)
1. Limestone	34	30-2,005	346[1]	1-1,700	177.0[2]
2. Carboniferous sedimentary rocks	480	38-1,500	292	0- 500	32.7
3. Triassic sedimentary rocks	553	25-3,700	199	0- 760	19.8
4. Crystalline rocks[3]	799	27- 850	155	0- 180	14.3
a. Granite	70	60- 770	231	0- 180	37.7
b. Granite gneiss	156	27- 447	175	1- 100	11.7
c. Schist	37	31- 850	125	1- 150	10.3
d. Gneiss	211	35- 503	128	1- 150	9.0
e. Not identified	325	29- 789	150	0- 163	14.4
5. Triassic igneous rock	86	35- 500	130	1- 60	11.2
All rock units	1,952	25-3,700	201	0-1,700	22.7

[1]Median depth—241 feet.
[2]Median yield—19 gpm.
[3]Wells in sub-units a through e included in total for crystalline rocks.

median depth and yield of wells in limestone as shown in Table 1 may be more representative of the water-bearing capacity of the formation.

The Carboniferous sedimentary rocks of the Narragansett and Boston basins rank second among the major rock units in productiveness, and the Triassic sediments are a distant third. The comparatively large average yield of wells in these rock units may be attributed, in part, to well-developed systems of joints and to equally well-developed openings along bedding planes.

In order to obtain an idea of the comparative yield of the several types of crystalline rocks, wells that could be identified with subtypes are listed according to the four main groups into which the crystalline rocks in southern New England are divided—granite, granite-gneiss, schist, and gneiss. Not all of the wells penetrating crystalline rocks could be identified with a sub-type, and those not identified are included together as a fifth unit. The data show that

the average depth of wells in granite is much greater than in other crystalline rocks and that granite is by far the most productive aquifer, yielding more than three times the average for the other crystalline rocks. This would suggest that the number of fractures contributing to any one well is much greater in granite than in the other crystalline rocks and that the interconnection of vertical and horizontal joints is more continuous. The average yield of wells in granite-gneiss is considerably less than that of wells in granite and not much greater than the average yield from schist or gneiss. The lower yield in these rocks is probably due to the more irregular development and connection of vertical and horizontal joints.

In all cases but one it is seen that for rocks of a larger average yield there is a correspondingly greater average depth of well.

Based on the same data employed in Table 1, Table 2 is arranged to show the variation of yield with depth. When the data are considered collectively, it is seen that there is an average increase in the

TABLE 2.—AVERAGE YIELD, IN GALLONS PER MINUTE, OF WELLS WITHIN SPECIFIED LIMITS OF DEPTH ENDING IN THE SEVERAL ROCK UNITS

Rock unit	Number of wells	Depth, in feet				
		100 or less	101-200	201-300	301-500	More than 500
1. Limestone	34	7.9	12.3	138.0	238.5	516.1
2. Carboniferous sedimentary rocks	480	10.7	14.2	30.2	58.0	58.5
3. Triassic sedimentary rocks	553	10.5	12.7	30.1	63.4	161.3
4. Crystalline rocks[1]	799	12.5	12.5	18.8	20.4	39.3
a. Granite	70	37.4	37.0	54.7	26.8	43.5
b. Granite gneiss	121	9.5	9.7	13.7	26.9	——
c. Schist	21	6.4	9.2	4.5	——	——
d. Gneiss	162	7.6	8.0	9.4	15.8	——
e. Not identified	425	12.5	12.7	15.9	16.4	38.7
5. Triassic igneous rock	86	8.2	11.3	20.9	24.0	——
All rock units	1,952	11.1	14.2	25.2	38.1	58.2

[1]Wells in sub-units a through e included in total for crystalline rocks.

amount of water obtained with an increase in depth, for the limits of depth shown. Likewise, this same general increase with depth is noted in the yield of most rock types. This conclusion might have been reached in advance of the compiling of the data. As a well is drilled deeper, it intersects more water-bearing fractures, which add to its total yield. However, it is also remembered that the number and size of opening of these fractures decrease with depth. The problem then arises of determining the depth below which the cost of deeper drilling is not justified by the smaller increase in yield. The data in Table 2 would indicate that this limit is deeper than 500 feet in most rock units. It is believed, however, that the figures in the upper limits of depth are somewhat misleading. Drillers do not, as a rule, ascertain or record the number and the location of the water-bearing fractures intersected by a well. Thus, much of the water in deep wells may seep in from upper levels, and such wells should perhaps have been listed under a shallower-depth limit. Well drillers and others engaged in subsurface probing in southern New England have noted that there are certain limiting depths below which there is a decreasing probability of finding water-bearing fractures. These depths appear to vary with the type of rock. Some drillers report the limit to be about 250 to 300 feet in crystalline rocks and about 500 feet in Triassic and Carboniferous sedimentary rocks.

A third table has been prepared for the same data. Table 3 shows the percentage of wells within specified limits of yield in the various rock types. It is to be noted that only 3% of the wells inventoried were complete failures or yielded less than 1 gallon per minute. Likewise, only 10% of the wells yielded more than 50 gpm. The largest percentage of wells is in the range of yield of 1 to 20 gpm, and more than 50% of these yield 1 to 10 gpm. The fewest failures are in limestone and in Triassic sedimentary and igneous rocks. The greatest number of failures appear to be in the crystalline-rock group.

Topography

A third factor to be considered is the effect of topography on the yield of wells in southern New England. Well drillers in the area and drillers and geologists in other sections of the country indicate that the yield of many wells in valleys or plains is greater than

TABLE 3.—PERCENTAGE OF WELLS, WITHIN SPECIFIED LIMITS OF YIELD, IN THE SEVERAL ROCKS UNITS

Rock unit	Number of wells	Yield (gallons per minute)					
		Dry or less than 1	1 to 5	5 to 10	10 to 20	20 to 50	More than 50
1. Limestone	34	0	24	6	29	12	29
2. Carboniferous sedimentary rocks	480	3	27	18	13	20	19
3. Triassic sedimentary rocks	553	1	22	32	23	12	10
4. Crystalline rocks[1]	799	4	38	21	20	13	4
a. Granite	70	3	13	14	14	39	17
b. Granite gneiss	121	3	42	25	15	12	3
c. Schist	21	0	28	57	15	0	0
d. Gneiss	162	6	44	24	22	3	1
e. Not identified	425	3	39	19	22	13	4
5. Triassic igneous rock	86	1	35	34	19	9	2
All rock units	1,952	3	30	24	19	14	10

[1]Wells in sub-units a through e included in total for crystalline rocks.

that of wells on hills. The most probable reasons for this relation have been stated by Herrick and Le Grand[1] as follows:

(a) Less surface runoff and consequently more direct influent seepage occurs in valleys, owing to the flatter terrain.
(b) Influent seepage occurs from upland surface runoff.
(c) Influent seepage occurs from upland rock slopes beneath the residual material.
(d) Greater total void space is available for storage of ground water, as a greater thickness of residual material is commonly present.
(e) Many valleys are associated with faults and other parting planes, along which water circulates.

The figures in Table 4 appear to bear out this relation. It is shown that the average yield of wells in valleys and on plains is

[1]Herrick, S. M., and Le Grand, H. E. Geology and ground-water resources of the Atlanta area, Ga. Georgia Geol. Survey Bull. 55, pp. 11-12, 1949.

TABLE 4.—AVERAGE DEPTHS AND YIELDS OF WELLS IN VARIOUS TOPOGRAPHIC LOCATIONS

Topographic location	Number of wells	Average depth (feet)	Average yield (gpm)
Valleys	71	339	91.9
Plains	315	331	57.5
Slopes	503	198	18.8
Hills	138	188	7.7
Not identified	1,063	155	9.6
All locations	1,952	201	22.7

considerably greater than that of wells on hills. Furthermore, the average yield of wells in valleys and on plains is higher than the average yield of all wells in all locations.

Depth and Type of Overburden

The bedrock or "ledge" in southern New England is overlain in most places by a varying thickness of unconsolidated material. Predominantly *all* this material is glacial drift, a deposit formed during the advance or the wasting of the ice sheet that once covered New England. The upland areas are commonly covered by till, a heterogeneous deposit of sand, boulders and clay. Till is commonly known to drillers as "hardpan". As the rock materials making up the till are usually poorly sorted, water passes through it rather slowly. The valley areas are underlain by the washed material or outwash deposited by the melt-water of the ice sheet. As a result of the washing action, beds of sand and gravel were deposited in terraces or fans along the edges of the melting ice lobes in valleys or were spread out in broad plains over lowlands. The finer particles were partly removed and the remaining materials were sorted in varying degrees to form a more or less highly permeable deposit. However, where sizable lakes of melt-water were created, as in the Connecticut River valley, the finer material accumulated as clay or silt, which has low permeability.

The type and thickness of this overlying layer of drift may affect the yield of wells in the underlying rocks. The drift serves as a storage reservoir for precipitation. If permeable, it may transmit

large quantities of this water downward to the fractures of the rock, especially when a well is pumped. If highly impermeable it may limit the amount of available recharge to these fractures. The available data show, in Table 5, that the average yield

TABLE 5.—AVERAGE YIELD, IN GALLONS PER MINUTE, OF WELLS ENDING IN TRIASSIC SEDIMENTS BELOW SPECIFIC TYPES AND LIMITS OF THICKNESS OF GLACIAL DRIFT

Type of drift	All wells	Thickness of drift, in feet				
		15 or less	16 to 30	31 to 60	61 to 100	101 or more
Sand and gravel	13.0	11.6	13.0	13.4	12.8	13.5
Clay	13.6	8.7	15.7	16.5	10.8	10.9
Till	10.4	9.9	9.9	12.2	9.3	9.3
All glacial drift		10.5	11.6	13.8	10.9	12.0

of wells in Triassic sediments overlain by stratified sand and gravel is 13 gpm; where the sediments are overlain by clay it is 13.6 gpm; and where they are overlain by till it is 10.4 gpm. Thus there seems to be but little difference in the average yield of Triassic sediments mantled by the several types of overburden. Wells in rock lying beneath till show a slightly smaller average yield. This lower yield may be due to the less permeable nature of the till, which increases surface runoff, thereby making less water available for movement into cracks in the bedrock. Or again, it may be due to the topography, till commonly occurring in upland areas. The highest average yield is for wells beneath clay. Clay is of low permeability, like till, so that the higher average may reflect mostly the topography, because the clay occurs principally in valleys.

It is not altogether surprising that wells in rock below the several types of overburden do not show a greater difference in average yield, when one considers the difficulties of isolating the effect of the type of overburden from the other factors. No matter how permeable the overburden may be or how much water is stored in it, the rate at which this water is transmitted to the well depends upon the number and the size of openings of the fractures in the bedrock at its upper surface. Where the capacity of the overburden to transmit water

is exceeded by the intake capacity of the bedrock in the vicinity of the well, then the former can have an important bearing on yield. In this case the yield of the well is dependent upon the average permeability of the overburden in a vertical direction. Under these conditions it might be expected that wells in rock lying below the more permeable sandy gravel deposits would yield larger supplies of water.

The data in Table 5 are arranged to show the variation in average yield of wells in Triassic sediments with thickness of the three types of glacial drift. It can be seen that in general the average yield changes little with increasing thickness of overburden. Again this might have been anticipated, as the thickness of unconsolidated material is of secondary importance to the permeability. An increased thickness means an increased storage capacity, but not necessarily an increased water-yielding capacity in the underlying rock.

Location of Wells

It is impossible to compose a set of rules for choosing the most profitable location for a well on a given property. Where there is limited space to drill a well, convenience determines the location. However, the well should be as far from contaminating influences as possible. Where there is a choice of locations several factors should be considered.

(1) Where more than one type of rock underlies the property, a well should be located in the type that is known to yield the larger amount of water, on the average. For example, where Triassic sediments adjoin crystalline rocks along the margins of the Connecticut River valley in Connecticut and Massachusetts, wells should be located in the larger-yielding Triassic material.

(2) Because larger yields are obtained from wells in valleys where glacial drift is thickest, this location should be chosen in preference to a site on a slope or hilltop.

(3) There is some chance that a larger yield may be obtained where sandy gravel deposits overlie the rock; therefore, wells should be drilled through these deposits rather than through clay or till. Of course, where a sandy gravel deposit attains some thickness, the deposit itself should be explored for water instead of the bedrock. This is sometimes ignored in drilling operations.

Depth of Wells

Though the wells range in depth from 25 to 3,700 feet, few are more than 500 feet deep. About 80 wells out of 100 are between 50 and 250 feet, though the depth varies with the type of rock penetrated. For maximum yield, an average depth somewhat greater than 200 feet is indicated.

Yield of Wells

There is always a chance that a rock well will not obtain the yield desired from it. The average well in southern New England yields 22.7 gpm. Only 3% of the wells fail to yield 1 gpm, but only 10% yield over 50 gpm. About 73 wells out of 100 yield between 1 and 20 gpm. In general there is an increase in yield of wells with increase in depth, but a decreasing probability of finding water-bearing fractures below certain depths is indicated for certain rock types. In crystalline rock this limiting depth appears to be about 250-300 feet; in Triassic and Carboniferous sedimentary rocks, about 500 feet. Thus if a well has reached a depth of 300 feet in crystalline rock and no water-bearing fractures have been struck, the chances of finding water are better if a second well is begun in a new location. Where a well has progressed to these depth limits and is yielding a small but insufficient amount of water, the yield may often be increased by modern methods of development. Techniques such as surging and wire-brushing and the use of dynamite and dry ice are reported to improve the yield of a well, sometimes by as much as 100%.

GROUND-WATER STUDIES IN RHODE ISLAND*

BY WILLIAM B. ALLEN**

[*Read April 17, 1952.*]

ABSTRACT

Ground-water studies conducted by the U. S. Geological Survey and State agencies as a coöperative program since 1944 show that the most prolific sources of ground water in Rhode Island are the deposits of sand and gravel located along rivers and in preglacial, buried valleys. Such deposits are found in the eastern part of Providence and Kent Counties and scattered throughout Washington County. Yields of 2,700 gpm have been reported from the glacial deposits—five times the best yield reported from the bedrock formations. Ground-water surveys of Rhode Island show that an estimated average of about 25 mil. gal. of water is pumped daily from wells in the State. On the basis of an average annual precipitation of about 39 in. and an area of 1,058 sq. mi., the amount of water reaching the State is about 700 billion gallons yearly on the average, or about 1,900 mgd on the average. This amount is many times the present ground-water pumpage, but the actual availability of this water for such pumping depends on how much of it can and does percolate to the various and several ground-water reservoirs. It is expected that the more detailed studies planned and in progress for different parts of the State will afford answers to that question.

Ground-water supplies are found in underground reservoirs or "aquifers." Such aquifers store water and serve as pipe-lines or filters for the movement of water from points of higher pressure to points of lower pressure. A knowledge of their functions is essential for balancing withdrawal and recharge.

Recent methods of increasing ground-water supplies are induced infiltration from streams and artificial recharge. Both devices are

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common in the New York-New England area, but are used only occasionally in Rhode Island.

Problems of overdevelopment involve the reservoir and pipe-line function of aquifers. Such problems are commonly corrected by relocation or more adequate spacing of wells.

Contamination of ground water by unusable surface water and brackish sea water is highly important. In Rhode Island rapid strides in the abatement of stream pollution are being taken by the Blackstone Valley Sewage Authority, the State Department of Health and allied agencies. An inflow of salt water from Narragansett Bay has occurred in Providence, Warren and East Providence. Corrective measures suggested are to build up the head of ground water by reduction in pumping, by spacing wells farther apart, or by artificial recharge.

Introduction

This paper has been prepared with the idea of presenting a general over-all view of the ground-water resources in Rhode Island and discussing geologic and hydrologic problems that affect the yield of ground-water reservoirs. Included also are some of the more recent ideas in the field of location, appraisal and development of ground-water resources, which are receiving more and more consideration with the passing years, because of the greater emphasis on development of ground-water supplies.

The over-all purposes of the ground-water studies conducted in Rhode Island by the Ground Water Branch, U. S. Geological Survey, are (1) to ascertain areas capable of supplying large quantities of water for industrial and municipal development, (2) to determine which of the readily available supplies are not being utilized fully, (3) to provide data leading to improved development and recovery of ground-water supplies, and (4) to measure and study the trend of water levels in observation wells, in order to determine the balance between replenishment of the underground supply and the amount withdrawn artificially or discharged naturally.

The work of the Ground Water Branch and geologic investigations by the Geologic Division are being made concurrently in Rhode Island in coöperation with the State Development Council. The Sur-

face Water Branch of the Geological Survey has a coöperative agreement with the State Department of Public Works, Division of Rivers and Harbors, to gage the flow of the principal rivers of Rhode Island. The original plan for ground-water studies called for the collection of information for individual areas corresponding to the U. S. Geological Survey 7½-minute topographic map. Since the start of this phase of the program, reports have been published for the Pawtucket and Georgiaville quadrangles and field work has been completed in the East Greenwich, Bristol and Crompton quadrangles. In addition to these studies, comprehensive reports on ground-water resources of the cities of Providence and Woonsocket have been published, and less extensive studies have been made for several critical areas, such as Bristol County, Exeter, Mashapaug Pond, Goddard Park, and other localities (Fig. 1).

In 1950 it was considered advisable to assemble for evaluation the many available data on ground and surface water for the entire State before continuing further with the detailed studies on a quadrangle basis. Thus an over-all picture of the water resources could be obtained, to determine areas where ground-water supplies could be utilized more fully. For this purpose a reconnaissance type of report was prepared, which will be released soon.

Using data collected in the reconnaissance report, State and Federal coöperating officials reëxamined the ground-water project in regard to completing the detailed inventory of ground-water resources by quadrangles within the next 5 years. It was decided that a considerable saving in both time and money could be made by conducting studies in groups of quadrangles at one time rather than in one quadrangle. Such groups of quadrangles are shown in Figure 2. The first group project covers the northeast area or the Providence, East Providence, East Greenwich, Bristol and Fall River quadrangles. The grouping of quadrangles in other areas to be studied may be changed according to future needs for ground-water data.

A very important part of ground-water investigations is the determination of the trend of water levels in observation wells. The location of the 53 observation wells currently in operation in Rhode Island is shown in Figure 3. Ground-water levels have been measured periodically since 1944 in several wells at Providence, since 1946

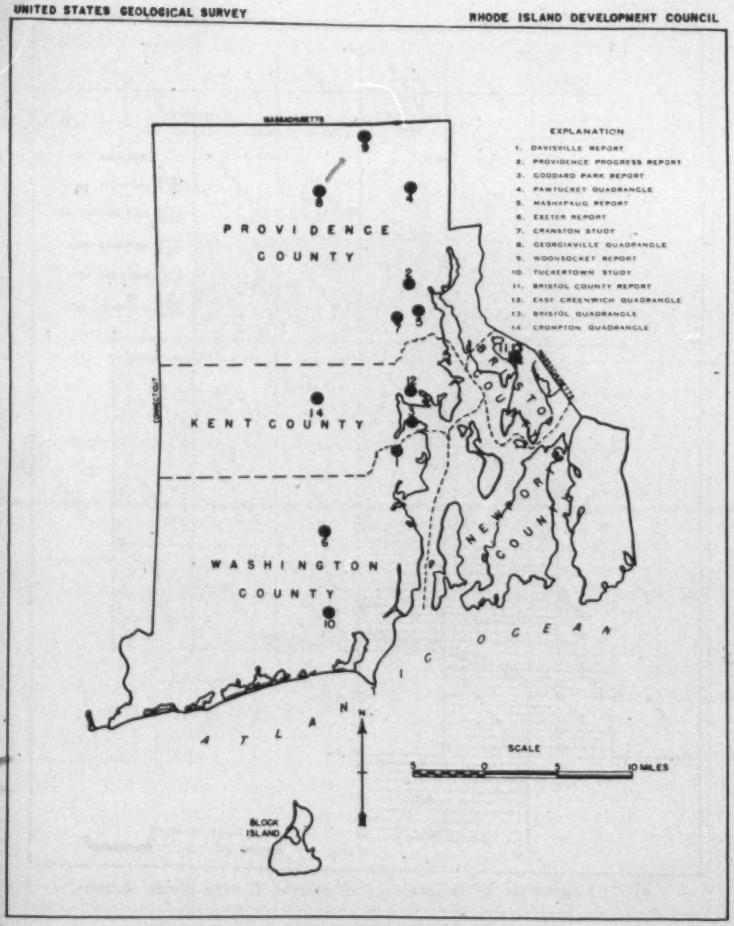

FIG. 1.—AREAS COVERED BY GROUND-WATER STUDIES.

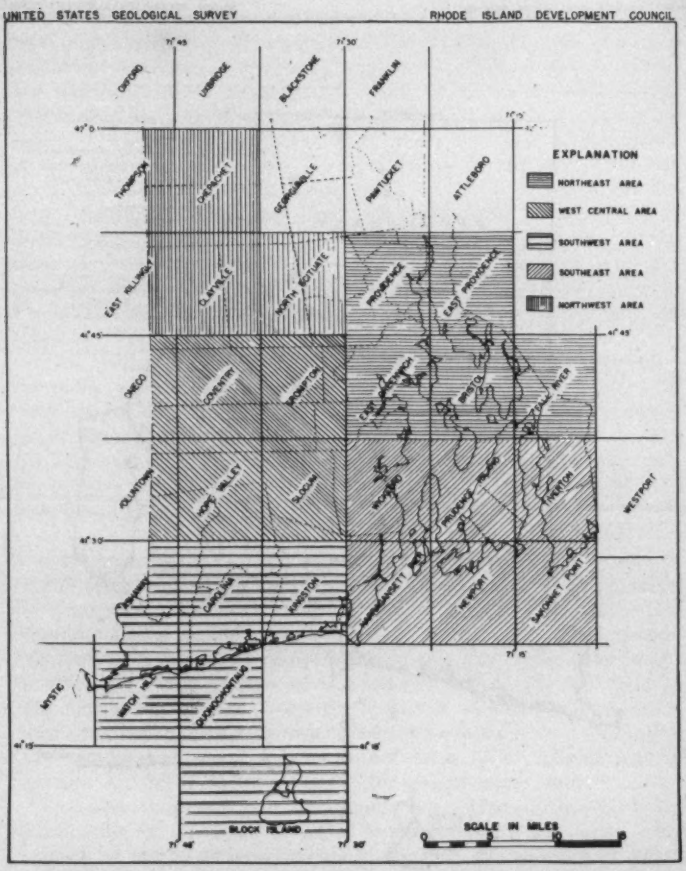

FIG. 2.—GROUPING OF QUADRANGLES IN FUTURE GROUND-WATER REPORTS.

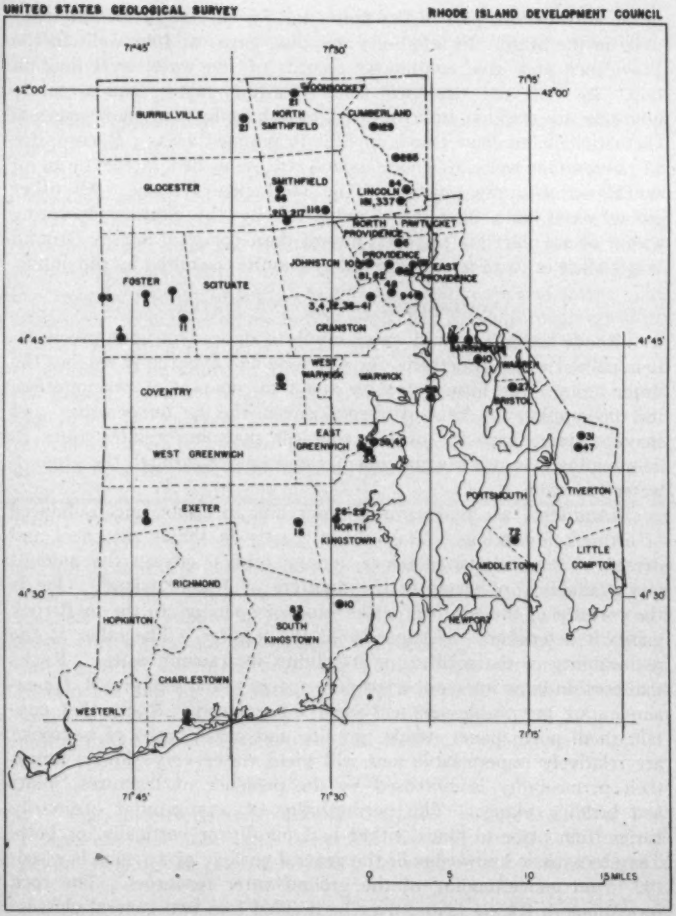

FIG. 3.—WATER-LEVEL OBSERVATION WELLS.

in the northeastern part of the State, and for shorter periods at other wells in the State. In addition, recording gages at four wells in the Providence area give continuous records of the water-level fluctuation. In wells not equipped with recording gages, manual measurements are made at intervals of 1 month or less, to show seasonal fluctuations or to show trends in heavily pumped areas. Records for all observation wells in Rhode Island are published in the series of annual water-supply papers of the Geological Survey. All other ground-water data have been published by the State coöperating agency or are placed in the open files of the Geological Survey Ground Water office in Providence, where they may be consulted by the public.

Availability of Ground Water

Rhode Island's ground-water supply is dependent upon precipitation, part of which penetrates the soil zone and eventually reaches the water table. The form, intensity and total amount of precipitation, the topography, the kind of vegetal cover, the air temperature, and the absorptive capacity of the soil—all play interrelated rôles in determining how much water reaches and passes through the ground-water reservoir.

"Aquifers" are underground reservoirs in either unconsolidated or bedrock formations. They differ greatly in shape, thickness and extent. Two important factors, among others, govern the amount and availability of water in the aquifers of Rhode Island. One is the porosity of the aquifer, or the ratio of openings to the total rock mass; it determines the capacity of the aquifer. The other is the permeability of the aquifer, or its ability to transmit water. Rocks that contain large interconnected pore spaces (sand and gravel, for example) are permeable and will yield water freely. Rocks that contain small pore spaces (shale, granite and other types of bedrock) are relatively impermeable and will yield water very slowly, unless their permeability is increased by the presence of fractures, joints and bedding planes. The permeability of any aquifer ordinarily varies from place to place, either horizontally or vertically, or both. Therefore, basic knowledge of the general geology of a region is essential to an understanding of the ground-water resources. The rock formations in Rhode Island may be divided into two general classes:

the consolidated bedrock and the overlying unconsolidated surficial deposits. Ground-water conditions in these two groups are quite different.

Consolidated Rocks

The consolidated rocks or bedrock of Rhode Island consist of igneous, sedimentary and metamorphic rocks, which have been divided into a number of recognized geologic formations. However, as previously mentioned, the bedrock has low porosity and therefore a study of the various kinds of bedrock is of little significance. Of more importance are the fractures, joints and bedding planes, which control the yield and rate of recovery of ground water. These openings may be regular or quite irregular in occurrence and form. The factors that control storage and movement of water through openings of this type are topography, depth and type of overburden, and rock structures. Consequently the prediction, in advance of drilling, of the amount of water that may be pumped from bedrock is exceedingly difficult. Data on 787 bedrock wells in Rhode Island show that the consolidated rocks have produced about 22 gpm on the average and up to a reported maximum of 500 gpm.[1] The most successful rock wells are those that penetrate thick deposits of sand and gravel before entering the bedrock. Drillers of bedrock wells in Rhode Island seldom encounter water-bearing fractures below depths of 300 to 500 ft and therefore generally discontinue drilling if a suitable water supply has not been found by the time these depths have been reached. The yield of a bedrock well that is not supplying enough water may sometimes be increased by surging, swabbing, dynamiting, and use of dry ice in the well. The study of rock structures, both on the surface and underground, provides a basis for a more intelligent selection of well sites. Such studies include mapping of surficial deposits, examination of rock cores and electric logging.

Unconsolidated Deposits

The unconsolidated surficial deposits, largely resulting from Pleistocene glaciation, are composed of sand and gravel, and till. About 85% of the State's surface is covered with a veneer of glacial

[1]Wells that are reported to yield more than 200 gpm are believed to be withdrawing water from the overburden.

till, a heterogeneous mixture of clay, sand and larger rock fragments, the composition of which differs from place to place. In general, wells finished in till yield less than 2 gpm unless they also penetrate associated stringers of sand or gravel. Although much of the water pumped from farm and household wells is obtained from till, this material is not recommended as a source of moderate to large supplies of water.

The unconsolidated deposits of sand and gravel are the most productive aquifers in Rhode Island because of their high porosity and permeability. The development of large ground-water supplies for municipal and industrial uses is restricted to areas underlain by sand and gravel. Extensive deposits are situated along rivers and in buried valleys, but many small aquifers are scattered elsewhere in outwash plains and deposits. About 15% of the State's surface is covered by areas of promise for moderate to large ground-water development. These areas are shown in Figure 4. For example, in Providence and Kent Counties, areas of promise are largely restricted to the eastern part of the State. In Washington County such areas are scattered. In Bristol County they are located principally in Barrington. In Newport County there are no areas of promise.

Ground water in the sand and gravel aquifers is contained in and moves through pore spaces between individual sand grains and therefore is usually not limited to narrow channels, as it is in bedrock. In Rhode Island no large-scale quantitative tests of these aquifers have been made, but detailed quadrangle studies in the northeastern part of the State have shown that aquifers in this area undoubtedly are far from being fully utilized. The same conditions are believed to exist in other parts of the State, except in a few scattered areas where wells have been placed too close together or where excessive pumping has drawn in salt water or surface water of undesirable quality.

The yield of a well penetrating a sand and gravel aquifer is difficult to determine in advance, for such aquifers differ widely in permeability within relatively short distances, both horizontally and vertically. Yields of such wells range from 3 to 2,700 gpm and average about 250 gpm. Expressed in the more convenient term of specific capacity, or yield per foot of drawdown, the yield of these aquifers

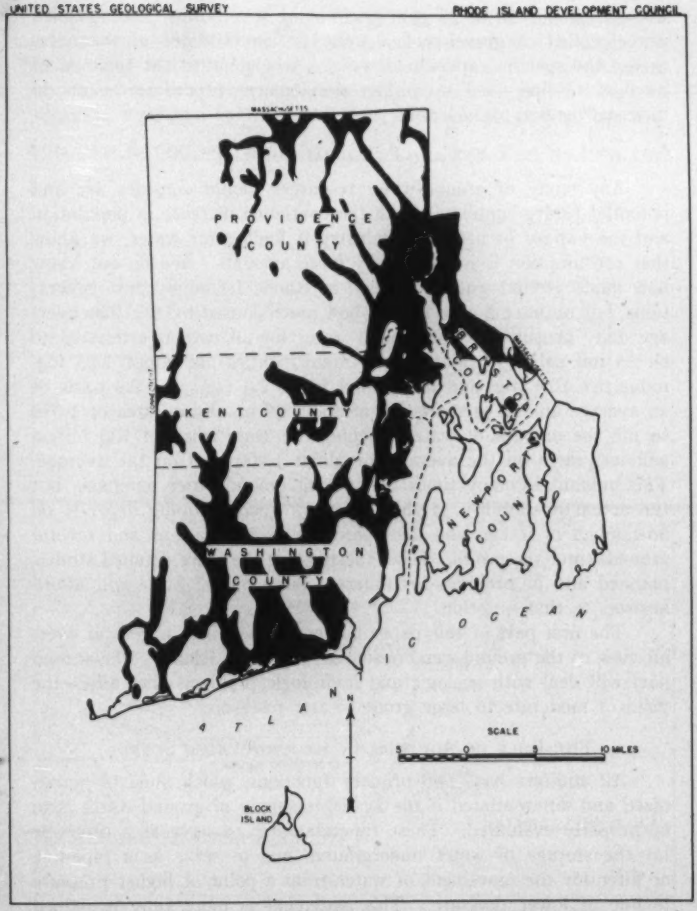

FIG. 4.—AREAS OF PROMISE FOR MODERATE TO LARGE GROUND-WATER DEVELOPMENT.

averages about 20 to 25 gpm per foot of drawdown. The so-called gravel-walled or gravel-packed wells, or modifications of them, increase the specific capacity of wells. At some sites the removal of medium to fine sand by proper development procedure results in "natural" gravel-packed wells.

Use and Potential Supplies

Any study of ground-water resources should consider use and potential future supplies. With the continued increase in population and the variety of uses that industry is finding for water, we know that consumption is increasing by large amounts. We do not know how much ground water was used in Rhode Island by past generations, but we have a good idea of how much is used today. The average daily consumption of ground water for all uses is estimated to be 25 mil gal. Of this amount, municipalities use about 12.3 mg, industries 10.4 mg, and homes and farms 2.3 mg. On the basis of an average annual precipitation of about 39 in. and an area of 1,058 sq mi, the amount of water reaching the State is about 700 billion gallons yearly on the average, or about 1,900 mgd on the average. This amount is many times the present ground-water pumpage, but the actual availability of this water for such pumping depends on how much of it can and does percolate to the various and several ground-water reservoirs. It is expected that the more detailed studies planned and in progress for different parts of the State will afford answers to that question.

The first part of this paper has been devoted to a general overall view of the ground-water resources in Rhode Island. The second part will deal with geologic and hydrologic problems that affect the yield of moderate to large ground-water reservoirs.

The Rôle of Aquifers in Ground-Water Supply

All aquifers have two primary functions, which must be recognized and differentiated if the available supply of ground water is to be properly evaluated. These functions are: to serve as a reservoir for the storage of water underground, and to serve as a pipe-line or filter for the movement of water from a point of higher pressure to one of lower pressure. This statement is more fully explained

by Figure 5, which is a generalized diagram of a typical aquifer under natural conditions in a Rhode Island river valley. The diagram is generalized in that only one aquifer is shown and it is assumed to be composed of porous and permeable sand and gravel. As this idealized condition rarely exists, it is advisable to give serious con-

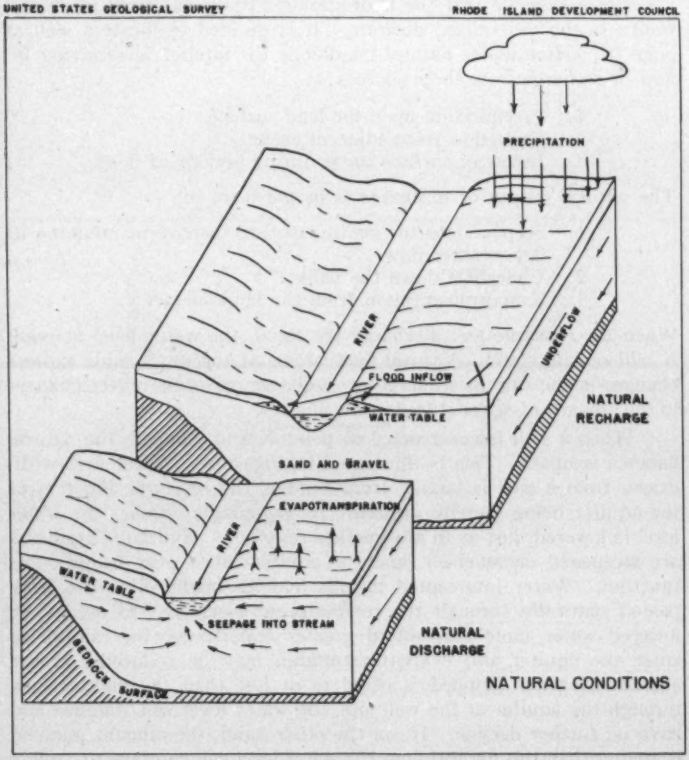

FIG. 5.—DIAGRAM OF TYPICAL AQUIFER UNDER NATURAL CONDITIONS.

sideration to a full-scale hydrogeologic investigation before large-scale ground-water developments are made. Such a study might involve some or all of the following: geologic mapping, test drilling, geophysical probing, pumping tests, chemical analyses of water samples and quantitative studies of the amount of water available.

Let us assume that the hydrogeologic study has been made and return to the generalized diagram. It is decided to locate a well at point X. Here under natural conditions the aquifer is receiving inflow or recharge from three sources:

1. Precipitation upon the land surface.
2. Underflow from adjacent areas.
3. Inflow of surface water during periods of flood.

The natural output or discharge is in the form of:

1. Seepage into the nearby stream, thereby maintaining its dry-weather flow.
2. Underflow down the valley.
3. Evapotranspiration from the land surface.

When the recharge and discharge are equal, the water level at point X will remain static. Natural fluctuations of the water table express changes in the rate of recharge or discharge and thus reflect changes in the amount of water stored in the aquifer.

When a well is constructed at point X and pumped, the natural balance is upset. This is illustrated in Figure 6. Water first withdrawn from a well is largely from storage, the reservoir function of the aquifer being thereby utilized. As pumping continues the water level is lowered, just as in any surface reservoir. Hydraulic gradients are steepened or reversed, and the aquifer takes over its pipe-line function. Water intercepted in this manner might otherwise have passed naturally through the reservoir and been lost to use. The lowered water table may afford greater opportunity for rainfall to enter the aquifer and evapotranspiration may be reduced. If the amount of water pumped is equal to or less than the natural flow through the aquifer at the well site, the water level will stabilize and have no further decline. If, on the other hand, the amount pumped is greater than this natural flow, the water level will continue to decline

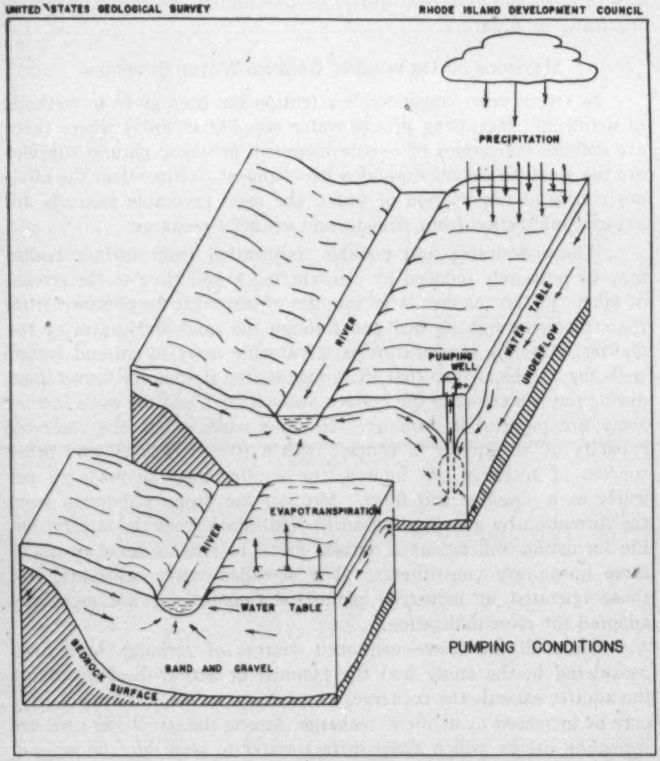

FIG. 6.—DIAGRAM OF TYPICAL AQUIFER UNDER PUMPING CONDITIONS.

and the cone of depression will spread until it reaches the stream. Thereupon some of the stream flow will be diverted to the well, filtering through the sand and gravel of the aquifer, as shown diagrammatically in Figure 7.

Methods of Increasing Ground-Water Supplies

In recent years considerable attention has been given to methods of artificially increasing ground-water supplies in areas where there are definite indications of overdevelopment or where natural supplies are too small to permit expanded development. Other than the obvious course of importation of water, the most favorable methods are induced infiltration from streams and artificial recharge.

Where desirable and possible, infiltration from surface bodies may be purposely induced by constructing a well close to the stream or lake. In this manner large supplies of water can be obtained from the stream through its bed and through the sand and gravel of the aquifer. As low temperature is a valuable asset of ground water, wells may be located so that those nearest the stream are drawn upon during the winter, when the surface water is cold, and the wells farther away are pumped in summer. In some cases where the reservoir capacity of an aquifer in contact with a stream is small and other sources of recharge are limited, the aquifer may be made to act solely as a pipe-line and filter. Most of the water withdrawn from the formation by a well will then be infiltration from the stream, but the formation will act as a natural filter, to remove some or all of some unsanitary constituents. The so-called water collectors, like those operated at industrial plants in Connecticut, are especially adapted for river infiltration.

When all the above-mentioned sources of recharge have been considered in the study and the planned or actual discharge from the aquifer exceeds the recharge, as a last resort the available supply may be increased by artificial recharge. Among the structures used are spreading basins, which allow surface water to seep into the ground, infiltration trenches, which drain water from a surface source and convey it to underground storage, and recharge wells, through which new or used water is added directly to the aquifer. The first two devices usually cannot be employed in built-up areas, but re-

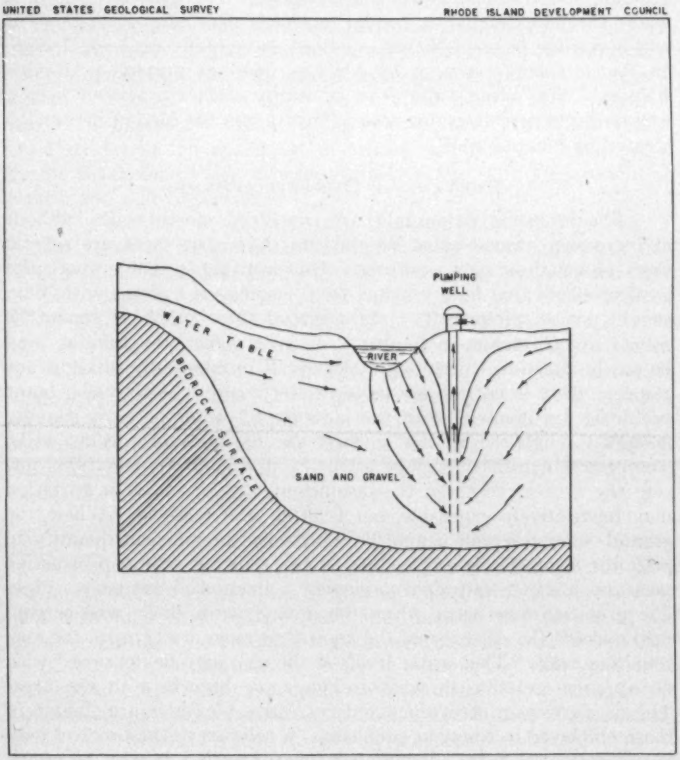

FIG. 7.—DIAGRAM OF TYPICAL AQUIFER SHOWING CAPTURE OF STREAM FLOW BY INDUCED INFILTRATION.

charge wells can be used anywhere that the underground hydraulic, geologic and economic considerations permit. In general, if ground water can be extracted, it can be replaced. Recharge through wells and spreading basins is being practiced successfully on Long Island. In Rhode Island recharge by wells is small at present. At New Britain, Conn., clear water from a nearby stream is spread over a well field and percolates downward directly into the cone of depression created by pumped wells.

Problems of Overdevelopment

The foregoing paragraphs are concerned mostly with methods of increasing ground-water supplies, to show that there are several ways of handling such problems. However, the actual or potential local problems that have resulted from insufficient hydrogeologic data should not be minimized. Only a few of these problems cannot be solved by one means or another. Often, however, the solution may be costly and time-consuming. Nearly all problems are alike in one respect; there is not enough usable water where it is wanted. Some problems are involved with the inability of water to move rapidly enough through the aquifer to meet the demands of specific wells. The reservoir problem may be solved by diversion of water from outside the area to recharge the insufficient supply. Such importation may be relatively expensive, but is often well justified. Where the ground-water reservoir cannot be replenished in sufficient quantity to meet the demands, by any method, a *real* shortage exists. Corrective measures are then restricted to outright reduction of pumpage. Pipeline problems may occur where the draft from a single well or well field exceeds the capacity of the aquifer to move water from the surrounding areas. Thus water levels at the well may be depressed with no apparent relation to what is happening elsewhere in the area. This is a common occurrence and corrective measures are similar to those employed in reservoir problems. A solution to the problem may be relocation, or more adequate spacing, of wells. It may be necessary to develop new well fields in the same reservoir but outside the areas affected by pumping of the older wells.

A serious problem results when a fresh ground-water supply is being contaminated by unusable surface water. Large-scale pump-

ing of wells in aquifers adjacent to streams will induce the inflow of surface water. The filtering action of the stream-bed sediments and of the aquifer itself removes bacterial impurity of the surface water. However, certain chemical contaminants, such as acids, cannot be filtered out and may ruin a good water supply if the concentration is gradually built up. The chemical quality of the original source of supply, the stream, is highly important. In Rhode Island rapid strides in the abatement of stream pollution are being taken by the Blackstone Valley Sewage Authority, the State Department of Health and allied agencies.

In a number of places in Rhode Island localized pumping of ground water in coastal areas has lowered the water table and has resulted in an attendant inflow of salt water from Narragansett Bay or tidal estuaries. Some industrial wells in Providence and Warren pump ground water contaminated with chloride concentrations as high as 3,000 ppm. One industry in East Providence has abandoned its wells for this reason. Corrective measures are again similar to those listed before. The head of ground water may be built up again through a reduction in pumping, by spacing wells farther apart, or by artificial recharge. It may be possible, in some cases, to build up the head by artificial recharge without reducing the rate of pumping, as has been done in some coastal areas.

Conclusions

In conclusion, it should be emphasized that because of the need for larger and larger supplies of ground water, all available supplies should be mapped adequately, and the quantity and quality of the water determined, before large-scale installations are made. Furthermore, once the installations are made it is important that systematic water-level observations and accurate inventories of the pumpage be maintained, in order to permit adjustments in the rates of withdrawal in an orderly and efficient manner before disaster strikes.

PUMPS AND PUMPING STATIONS

BY RICHARD HAZEN*

[*Read September 10, 1952.*]

Some of you have probably seen in the September, 1952, issue of the *National Geographic Magazine* the colored photograph of a wood and bamboo treadmill for pumping irrigation water in Indo-China. It is captioned "Father and Son Wearily Pedal Humanity's Revolving Squirrel Cage, but Get Nowhere." The father appears resigned to the chore; his son is scowling like any American boy confronted with a lawn to mow or garden to weed.

Irrigation is not needed in most of New England and the geography provides gravity water works in a great many towns. In fact, nowhere have greater efforts and care been taken to build reservoirs and pipe-lines that will assure gravity flow. However, as the towns have extended to the outlying and usually higher lands, pumping of part or all of the water frequently has become necessary. Fortunately, pumps, efficient in operation and requiring little attention, are available.

This paper is not concerned with the details and theory of pump design. These aspects are of vital importance, but usually to the pump manufacturers. We shall discuss here the several types of pumps available, advantages and disadvantages, and a few applications. The growing popularity of vertical pumps warrants particular attention.

The cost of pumping water is relatively less today than it was 30 or 40 years ago. The improvement has been due chiefly to the development of the centrifugal pump, which in turn was dependent upon efficient high-speed motors, steam turbines and engines. The direct-connected reciprocating steam pump is an efficient pump because there are no intermediate losses, such as in the generation and transmission of power. The steam turbine and diesel or gasoline engine have the same advantages. Steam turbines are used fre-

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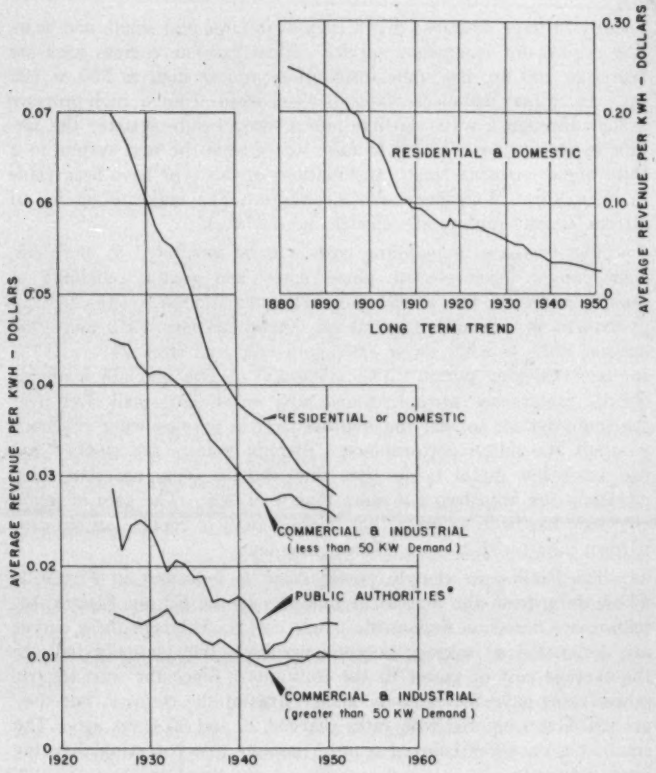
quently in large stations; diesel engines in large and small, and gasoline engines for emergency service. Most gasoline engines used are less than 100 hp, but some installations run as high as 500 or 600 hp. In a few instances water is discharged from a high-pressure system through a water turbine into a low-pressure system; the turbine in turn drives a pump to raise water from the first system to a third higher-pressure zone. Installations of this type have been made in New York, Waterbury and elsewhere. The majority by far of all centrifugal pumps are electric-motor-driven.

The decrease in pumping costs can be attributed to more efficient pumps, lower electric power costs, and greater reliability of pumps, requiring less attendance. One manufacturer cites the improvement in guaranteed pump efficiencies between 1910 and 1945 as from 65% to 82% for a 1,000-gpm unit, and from 74% to 87% for a 5,000-gpm pump. The efficiency of commercially-available electric motors has ranged around 90% in all but small sizes over the same period, so that the improvement in wire-to-water efficiency parallels the pump performance. Electric motors are smaller and use much less metal today than they did 30 years ago, but their efficiency has improved not more than 1 or 2%. The gain in pump efficiency has been so great that it often pays to replace an old centrifugal pump with a more efficient new one.

The decline in electric power costs is indicated in Figure 1. These data, from the Statistical Bulletins of the Edison Electric Institute, are based on nationwide power costs. Although these curves are designated as average revenue per kwh, they actually indicate the average cost of power to the consumer. Since the war, electric power rates have increased in many parts of the country, but they are still low compared with rates charged 20 and 30 years ago. The small consumer has enjoyed a much greater cost reduction than the large consumer.

The effect of improved pump efficiencies and lower power costs upon the cost of pumping water is illustrated by the following example: A tabulation of pumping costs per mil gal for a 5,000 gpm pump and a 700-gpm pump in 1926 and in 1951 shows the 1951 costs to be 39% lower for the large unit and 50% lower for the small unit.



TRENDS IN THE
AVERAGE REVENUE PER KWH

*Other than street lighting

SOURCE: Statistical Bulletins, Edison Electric Institute

FIG. 1.

	Large unit 5,000 gpm vs.100'	Small unit 700 gpm vs. 100'
1926--Wire-to-water efficiency	71%	63%
KW required	133	20
KWH per mg	465	505
Power cost per KWH	1.5¢	4.5¢
Power cost per mg	$6.85	$22.75
1951—Wire-to-water efficiency	77%	71%
KW required	122	19
KWH per mg	418	453
Power cost per KWH	1.0¢	2.5¢
Power cost per mg	$4.18	$11.32
Decrease in cost per mg	$2.67	$11.43
% decrease in cost per mg	39%	50%

In general, the cost of horizontal pumps has increased by 25% since the war, vertical pumps by 10%, and the electrical equipment by as much as 75%, but these equipment costs are frequently less than one-third of the total pumping-station costs. It cannot be argued that pumping stations are cheap, but it is a fact that many decisions reached years ago to provide gravity flow might be reversed today in the light of pumping costs and the cost of installing alternate works.

The cost of power is often relatively unimportant, compared with the cost of supervision and attendance. This has resulted in the present trend toward automatic or semi-automatic stations that need a minimum of attention. Automatic controls are available for starting and stopping pumps, alternating operation, starting standby engines, etc., all actuated by water pressures, water levels, flows or a time clock. Whenever automatic controls are used, care must be taken to install rugged equipment, protected from moisture, dirt and external damage. Otherwise, they cannot be kept in order.

An outstanding example of a modern pumping arrangement is the system used to raise Colorado River water into the aqueduct of the Metropolitan Water District of Southern California. Each of the 5 stations now includes three 200-cfs (130-mgd) pumps and additional units are being added. The stations operate in series through storage reservoirs and raise the water a total of 1,600 ft. The maximum lift

is 441 ft and the minimum 144 ft. The pumps are single-stage, vertical pumps with 4,300 hp motors for the lowest head, and 12,500 hp motors for the highest-head units. Mr. Robert Diemer, General Manager and Chief Engineer, has summarized the operating requirements as follows:

> "The pumping plants are manned during the day shift only, except at Gene, the headquarters of the system, where one 'system operator' stands watch on each shift. The normal crew at each plant consists of seven men: station chief, station electrician, operator, machinist, maintenance mechanic, and two utility men. The Intake and Gene plants, however, being close together, are covered by one crew and one additional electrician.
>
> "At the four plants which are unattended during the evening and night shifts, one of the crew is assigned to 'standby' duty and his residence is connected to the system operator at Gene by telephone. In addition, an alarm in his residence sounds in case of any mechanical or electrical trouble at the plant. Tripping circuits in all plants are so connected that loss of power, interruption of oil flow to bearings, high temperature of bearings, electrical failure in the driving motors, or similar trouble will automatically shut down the motors and close the pump discharge valves."

In this instance failure of a pump would have no immediate serious effect on the water supply, because there is plenty of gravity storage between the pumps and the consumer. Where normal and emergency water requirements are met by direct pumping without storage, they are dependent upon continuous pump operations and full-time supervision may be needed. However in the face of rising labor costs, the possibility of substituting automatic safeguards for additional shift operators should not be overlooked.

Centrifugal pumps may be classified accrding to the type of runner or impeller. Radial and Francis types are most commonly used for high and medium heads; for lower heads and higher flows, a mixed-flow impeller with discharge partly radial and partly axial is used. For still lower heads and very large flows the axial-flow or propeller type may be most satisfactory. The axial-flow type strictly is not a centrifugal pump, in that there is no radial flow. However, it is driven by a high-speed rotating shaft and is generally considered along with the other types.

The head and pressure range up to a capacity of 10,000 gpm, or 13 mgd, for horizontal and vertical pumps of these four types is indicated on Figure 2. Larger pumps are frequently used in irriga-

CAPACITY IN G.P.M.

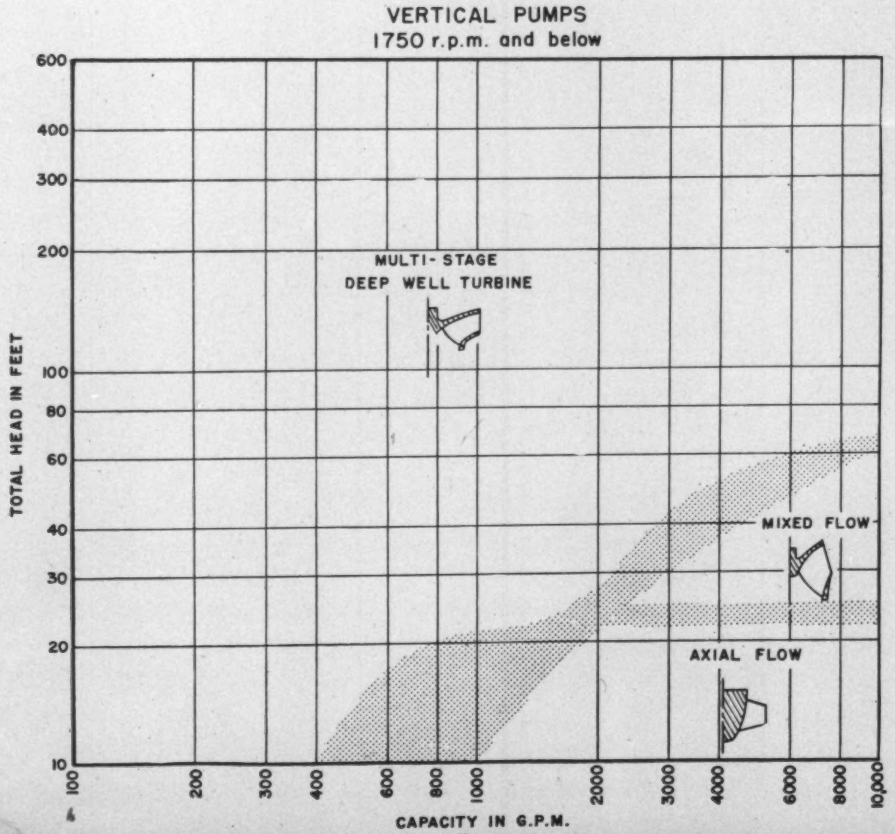
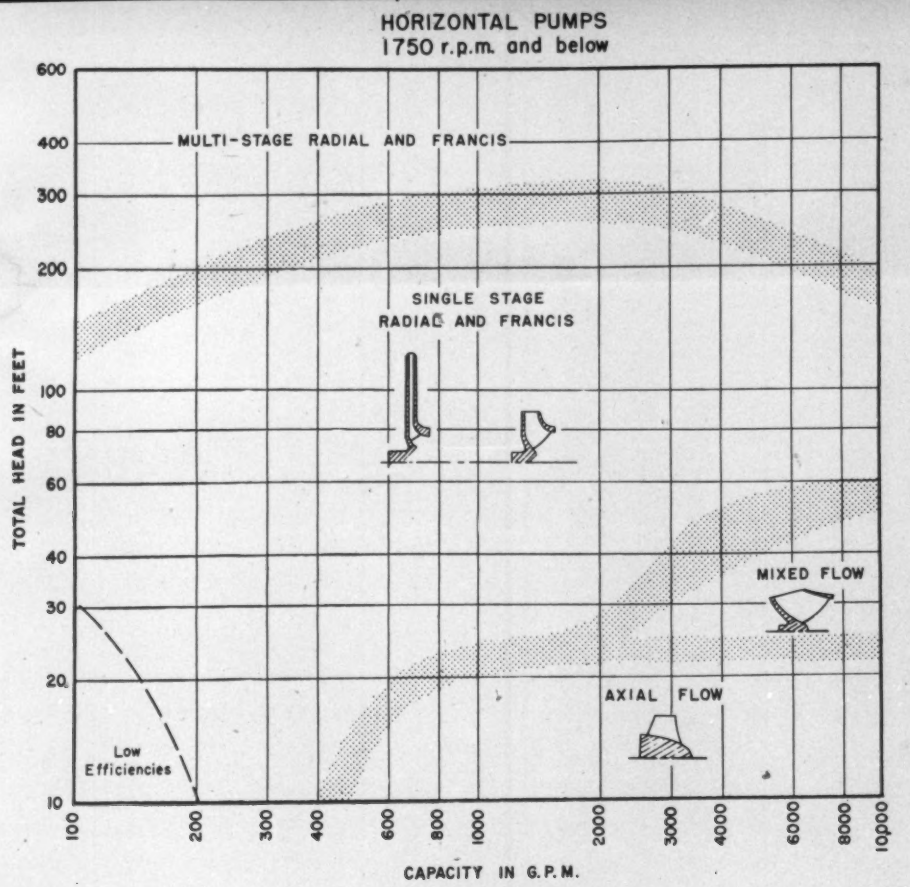


FIG. 2.


HORIZONTAL PUMPS
1750 r.p.m. and below
MULTI-STAGE RADIAL AND FRANCIS
SINGLE STAGE
RADIAL AND FRANCIS
MIXED FLOW
AXIAL FLOW
Low
Efficiencies
TOTAL HEAD IN FEET
600
400
300
200
100
80
60
40
30
20
10
100
200
300
400
600
800
1000
2000
3000
4000
6000
8000
10000
CAPACITY IN G.P.M.

tion and flood-control work, but most water-works pumps fall within the limits shown. Within these limits pumps of comparable efficiency can be obtained. With capacities of 1,000 gpm or less, it is hard to get a sustained pump efficiency in excess of 83 or 84%; in the larger sizes efficiencies up to 90% can be obtained. In comparing pump efficiencies, one should realize that there is a practical limit to the number of different pump casings and impellers each manufacturer can produce, and his pump may not meet a particular situation as well as one of his competitor's pumps. The variety offered by a manufacturer is usually a function of demand and the cost of patterns. Temporary high efficiencies often can be obtained by undercutting the vanes, to increase the opening for passage of water. The undercutting results in a thinner section at the tip of the vanes, which may corrode or wear within a short time.

The finishing of impellers and casings has a good deal to do with the efficiency. Time taken to file off a few rough burrs is well worth the cost. Performance tests at the factory and after installation are the best insurance against improperly finished pumps. A shop test may add 5% or more to the cost of the pump but is well worth while, except in purchasing very small pumps for which certified performance curves are generally acceptable. The cost of field tests after installation varies considerably with the complexity of the installation and the availability of accurate metering equipment.

In Figure 2, the range of application chart, there is considerable overlapping and the choice may not be clearly defined. For applications covered by the shaded portions of the chart, the best type of pump may be that shown on either side of the shaded section, the choice depending upon considerations other than the head and capacity desired. Pumps rarely operate at exactly the head and discharge rate for which they are purchased, and variations from the design point usually reduce the efficiency. The variation will be large or small, depending upon the type of pump and its characteristics. Figure 3, showing characteristic curves of the four types of pump, is taken from the DeLaval Steam Turbine Co. Handbook. These curves apply to pumps operated at a constant speed. Note the contrast between the flat head-discharge curve No. 4 of a radial-flow pump and the steep curve No. 1 of an axial-flow pump. A small in-

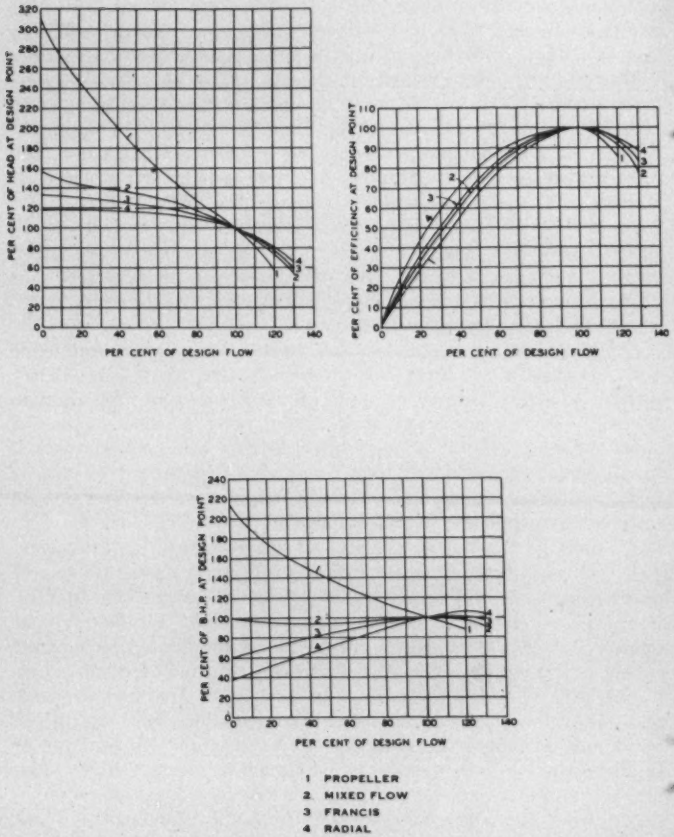

PER CENT OF HEAD AT DESIGN POINT
PER CENT OF DESIGN FLOW
PER CENT OF EFFICIENCY AT DESIGN POINT
PER CENT OF DESIGN FLOW
PER CENT OF B.H.P. AT DESIGN POINT
PER CENT OF DESIGN FLOW
1 PROPELLER
2 MIXED FLOW
3 FRANCIS
4 RADIAL

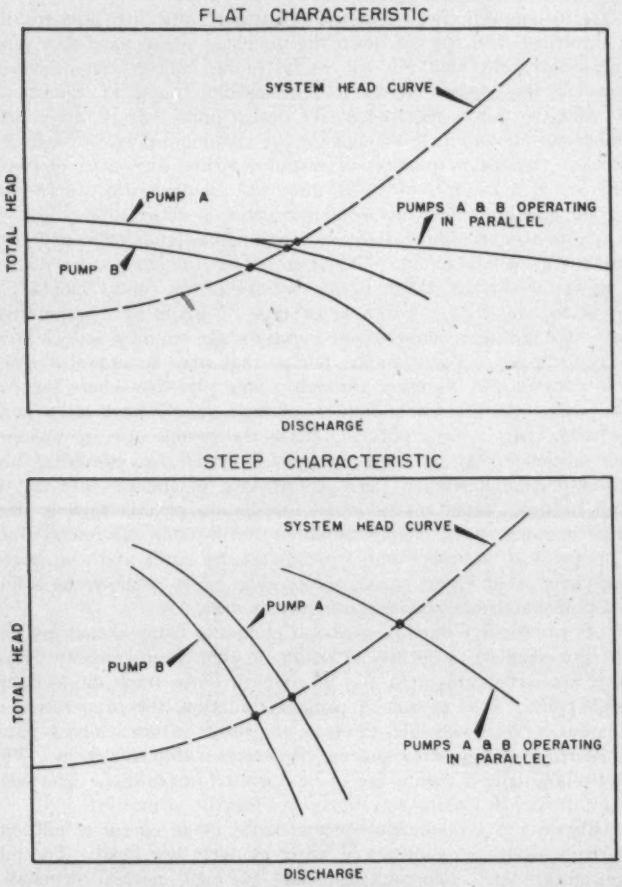
FIG. 3.—CHARACTERISTIC CURVES OF FOUR TYPES OF PUMP (FROM DELAVAL STEAM TURBINE CO. HANDBOOK).

crease in operating head, because of greater static lift, pipe friction, or throttling, will not cut down the discharge of an axial-flow pump appreciably, but may put the radial pump almost out of service. However, the power required for an axial-flow pump at shutoff may be twice the power required at the design point. It is rarely practical to use motors large enough for the maximum possible load, and, therefore, axial-flow pumps are installed either without a discharge valve or, if a valve is included, automatic controls are provided, to prevent starting the pump when the valve is closed.

Note also that the radial-flow pump has a flatter efficiency curve than the axial-flow pump. This means that if a pump must operate at flows other than design flows, the radial-flow pump should give the better efficiency. For a given type of pump all characteristic curves are not alike, and the best pump for the required service is not always selected. For example, pumps that may be operated singly or in parallel and discharge through a long pipe-line where the friction is an important part of the total head should have steep head-discharge characteristic curves. When the pumps operate together, their efficiency may be lower, but this is better than not being able to pump enough water. The performance of pumps with flat or steep head-discharge curves in an installation of this type is illustrated in Figure 4. It should be noted that a pump can operate only at the point of intersection of its head-capacity curve and t he system head curve. On Figure 4 the system head curve is shown as a line. In actual practice it is more frequently an area.

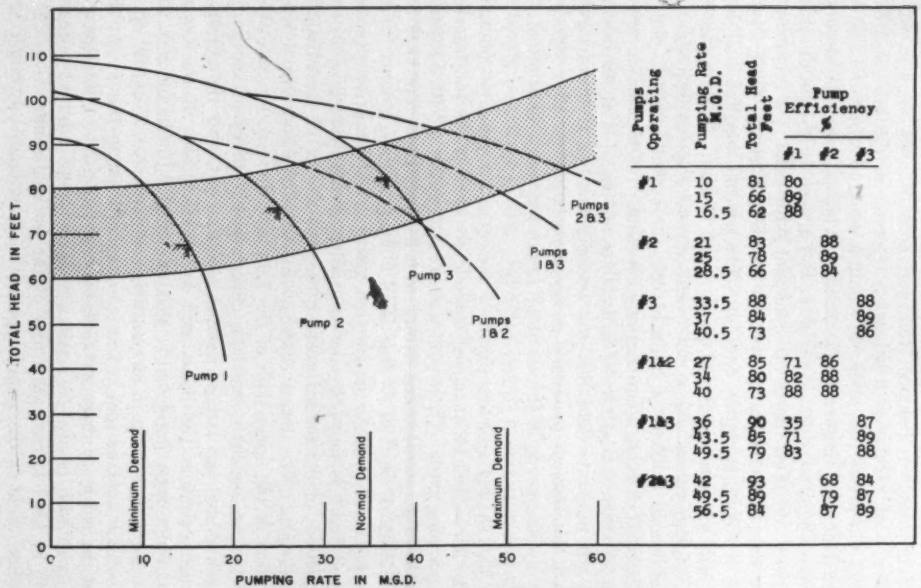
As previously stated, centrifugal pumps in many actual installations are required to operate at points on their head-capacity curves which are within this area and at some distance from the so-called design point. In analyzing a pump installation, the preparation of a system curve is valuable, to check the range within which a pump will be required to operate and the efficiencies within this range. This is especially true if pumps are to be operated in parallel. The steps to be followed in making a system curve may be of interest.

Figure 5 is a theoretical system curve, made up for a mill supply requiring large quantities of water at fairly low head. The mill requirements are: minimum demand—10 mgd, normal demand—35 mgd, and infrequent maximum demand—50 mgd.



PERFORMANCE OF PUMPS OPERATING IN PARALLEL

FIG. 4.



Pumps Operating	Pumping Rate M.G.D.	Total Head Feet	Pump Efficiency %		
			#1	#2	#3
#1	10	81	80		
	15	66	89		
	16.5	62	88		
#2	21	83		88	
	25	78		89	
	28.5	66		84	
#3	33.5	88			88
	37	84			89
	40.5	73			86
#1&2	27	85	71	86	
	34	80	82	88	
	40	73	88	88	
#1&3	36	90	35		87
	43.5	85	71		89
	49.5	79	83		88
#2&3	42	93		68	84
	49.5	89		79	87
	56.5	84		87	89

FIG. 5.—TYPICAL SYSTEM HEAD CURVES.

The supply is pumped from a river with a variation in water level of 5 ft. There is a balancing reservoir at the mill, and it is desired to operate this reservoir within a working range of 15 ft between high- and low-water elevations. The difference in elevation between high-water level in the river and low-water level in the reservoir is 60 ft. This represents minimum static head. The maximum static head is the difference between low-river level and full-reservoir level, and is 20 ft higher, or 80 ft. The transmission line is 54-inch pipe with a coefficient of 100 and is 9,000 ft long. As pumping station, piping, valves and a meter will introduce losses equivalent to an additional 1,000 ft of 54-inch pipe, the system curve is plotted on the basis of 10,000-foot transmission main. Friction loss increases from less than 1 ft at the 10-mgd rate to nearly 20 ft at the 50-mgd rate. It might be said that the transmission line is overdesigned. This is intentional, to reduce pumping costs and to have pipe-line capacity in reserve in the event of future increased demands at the mill.

The shaded area on Figure 5 indicates the range in pumping heads determined by changing water levels in the river and reservoir and pumping rates. Three pumps are provided: No. 1 with a capacity of 15 mgd at 66-foot head, No. 2 with 25 mgd at 78-foot head and No. 3 with 37 mgd at 84-foot head. Each of these pumps has an efficiency of 89% at the design point stated. The characteristic curve of each pump is shown by the solid line. The efficiency at the top and bottom of the shaded working range is shown on the tabulation to the right. The lowest efficiency is 80% for Pump No. 1 when operating at the upper limit of the zone.

The broken lines show the characteristic curves of the pumps when operated on various parallel combinations, and the efficiency of each pump at the upper and lower limits and in the central part of the operating range is also tabulated. It can be seen that the most serious drop in efficiency is for pump No. 1 at the upper part of the working zone, when operated with pump No. 3. At this point the efficiency drops to 35% because pump No. 1 is approaching its shut-off point. In actual operation pumps 2 and 3 probably would be used to cover this part of the working zone.

For each of the parallel pump combinations, the larger unit maintains a high efficiency throughout the operating range.

It can be seen that the curve for pumps 1 and 2 working together approaches the curve for pump 3 alone, so that they can carry the normal demand if pump No. 3 is out of service for repairs. However, pumps 1 and 2 will not deliver 35 mgd, nor will pumps 2 and 3 deliver 50 mgd if conditions indicated by the upper half of the operating zone exist. To obtain these flows during low river stages, it would be necessary to operate the reservoir at the mill below its normal flow line. The necessity for such operation would be infrequent.

The curves show that every increment of flow between 10 and 50 mgd cannot be obtained by the pumps or combination of pumps shown without some throttling at parts of the range. However, it is not necessary to pump at all possible intermediate rates between 10 and 50 mgd, as the reservoir at the mill can be used to "iron out" the differences between consumption and pumping rate. Actually, the combination of pumps and reservoir working together is self-balancing to a large extent. With the pumps operating alone, a 10-foot drop in reservoir level will increase delivery from 4 to 5 mgd and, with the pumps operating in parallel, a similar drop will increase delivery from 10 to 14 mgd.

It is quite probable that inefficient or unsatisfactory performance, found in some existing pumping stations, could have been foreseen and avoided if a carefully prepared and analyzed system curve had been developed at the time the pumps were purchased. Too often pumps are selected for the worst operating condition, which may occur less than 10% of the time. As a result, the pumps operate inefficiently 90% of the time.

In water-works operation the maximum efficiency of the individual pumps is rarely obtained, unless there is enough storage in the system or enough pumps of different sizes, so that each pump can be operated at its rated capacity. More often, a pump discharge must be throttled down, to create an artificial system head curve that will intersect the pump curve at the desired rate of delivery. This not only reduces the pump efficiency slightly, but also wastes entirely the head lost through the throttled valve. The total loss may be considerable.

The high-lift pumping station at Atlantic City, N. J., contains

four electric-motor-driven centrifugal pumps, rated at 2.5, 7.5, 13.0 and 17.2 mgd, to meet a winter daily demand of 5 mgd, a summer daily demand of 17 mgd, and peak hourly demands of 25 mgd. There is practically no storage on the distribution system. An analysis of the pumping records indicated that in spite of prompt shifting of pumps to meet fluctuating water demands, the average head loss from throttling was approximately 8% of the total pumping head.

With constant-speed pumps, best efficiency can be obtained only at or very near the design flow. A wider range can be obtained with multi-speed pumps, driven by steam turbines or diesel engines, because they can be speeded up to increase the capacity and head. The capacity increases proportionately to the speed, whereas the head increases as the square of ratio of the speed. For electric drives, multi-speed, wound-rotor, induction motors or constant-speed motors with hydraulic or magnetic couplings are available. Any of these involves a loss in efficiency of 5% or more and this loss should be compared to that caused by throttling a constant-speed unit. The higher cost of variable-speed equipment is not often justified, except in large low-head installations, where only a small change in head is a major percentage of the total head.

For the usual water-works installations, operating efficiency is best obtained by installing two or more pumps, frequently of different capacities, to be operated singly or in parallel, and delivering to a system where high-level storage is provided.

Vertical and horizontal pumps are basically the same. A multi-stage, deep-well turbine pump is simply a special arrangement of pump bowls or casings in series, usually with a radial-type runner. Axial-flow and mixed-flow pumps are made for both horizontal and vertical installations. The vertical pump is popular because it can be set down into the water and no suction piping, dry well or priming equipment is needed. The motor can be mounted at any convenient elevation above the pump. The vertical pump often simplifies station design and piping, saves space, and frequently eliminates valves on the suction side.

Placing the motor above the pump usually eliminates the possibility of flooding, either from high river stage or from a pipe break

within the pumping station. In well installations a weatherproof motor and starter are sometimes used without a well house. The pump need not hang in a well or in open water; the bottom bowl can be attached to a standard elbow or other pipe fitting, or water may be delivered to the pump at a higher elevation through a pressure casing, surrounding the bowls and column. In such cases the motor is usually close-coupled to the pump, with practically no discharge casing, and only enough height to clear the pump bowls must be provided. We have used this arrangement recently in the design of a ground reservoir and booster station, where there was plenty of head room in any event, and we wanted to keep the pump room as small as possible.

Vertical pumps are basically as efficient as horizontal pumps, although in some deep-well-turbine sizes, manufacturers' standard models are 3 or 4 points less efficient than horizontal pumps of the same capacity. If a well pump must be kept small in diameter to fit an undersized well casing, the velocities and friction losses are higher and the losses proportionately greater. On the other hand, a pump with large impellers and few stages may be more efficient as a smaller pump with more stages. Thus, for example, one manufacturer cites the performance of three standard pumps, each operating at 1,760 rpm, rated at 500 gpm vs. 200-ft head, with 100 ft of discharge column, as follows:

Well dia.	Pump bowl o.d.	Lift per stage	No. of stages	Pump efficiency
8"	7½"	17.5'	12	68%
10"	9½"	37.0'	6	81%
12"	11½"	66.0'	3	75%

An interesting point is that so many multi-stage well-type pumps are being used for purposes other than deep wells that manufacturers are developing lines not restricted to conventional well-casing diameters, and more efficient equipment is available, particularly in the large sizes. Probably the outstanding example to date of this type is the installation of six 5-stage 20-mgd pumps, designed for a total head of 600 ft at New York City's emergency Hudson River pumping station at Chelsea. The pump bowls are 36 in. in diameter, and the

discharge 24 in. in diameter. Each pump is driven by a 2,500-hp synchronous motor, operating at 900 rpm. The first bowls were light in some respects for the pressures involved and had to be replaced. Since then the pumps have operated satisfactorily. On test the pumps yielded wire-to-water efficiencies of 83 to 87%. Vertical pumps are being used in increasing numbers for river pumping stations, especially where there is a wide range between high and low river stages.

The high efficiency obtainable with axial-flow and mixed-flow pumps suggests the use of several stages for high-capacity, high-head installations. However, in practice the types of pumps are usually limited to two stages, because otherwise the shaft diameter to transmit the necessary power is so great as to restrict the water passages. Single-stage pumps of the radial or Francis type, with large-diameter impellers, are more satisfactory and, because of the large-diameter impeller, excessive speeds are not required even for high heads. The pumps of the Metropolitan District Aqueduct of Southern California, operating at 300 to 450 rpm with wire-to-water efficiencies of 87% or more, are of this type.

The principal objection to vertical pumps is that they cannot be disassembled for inspection, cleaning and repair as easily or quickly as a horizontal centrifugal pump, with casing split along the shaft. The vertical pump must be pulled out—in one piece, if short, or in sections, if long—and disassembled. Minimum clearness and adequate lifting facilities must be provided.

Vertical, hollow-shaft induction motors are readily available in standard sizes for most pumps. Vertical synchronous motors are harder to get in smaller sizes. For large installations the motors are especially made in any event and the type desired can be obtained. With engine or steam-turbine drive, a right-angle gear is necessary to drive a deep-well pump. This introduces another piece of machinery and some loss in efficiency. However, a geared speed changer is often required in any event, if a slow-speed diesel engine or high-speed steam turbine is used.

The bearings of vertical pumps may be oil- or water-lubricated. They are not as accessible as the bearings in a horizontal pump, but this is not ordinarily of much practical significance, because the

bearings in either type, if properly selected, are thoroughly reliable and require little attention. Particular attention should be given to thrust bearings and foundations in vertical pumps. The up-thrust in starting and the down-thrust in operation may be substantial.

No general rules can be stated for pumping-station design, unless they are to keep it simple and the piping big enough. The advantages of high efficiency in pumps can be lost entirely by a few unnecessary bends and lost velocity heads. Some pertinent points that should be kept in mind include:

(a) Need for standby capacity, in spite of industrial practice to provide none and the ability of many water works to operate throughout the war with little or no reserve. Standby capacity includes both pump capacity and motive power. If electric-service outages may occur, standby engine or steam drive is important. In some stations diesel-generator sets are provided. Although expensive, they are of value when pumping stations are operated in conjunction with water-treatment plants that have various auxiliaries dependent upon electric power.

(b) Provision for water hammer. Refer particularly to "Water Hammer Control" by S. Logan Kerr, in December, 1951, Jour. A. W. W. A. It has a handy and useful check list.

(c) Importance of suction-pit design for vertical pumps, especially of the mixed-flow and axial-flow types, to avoid high velocities and eddies. The wrong design will cut efficiencies surely and, if bad enough, it will reduce the capacity of the pump.

(d) Desirability of looped suction and discharge piping with shut-off valves, to permit operation of a part of the station if there is a pipe, valve or fitting failure. This safeguard is common in large stations; it is often omitted in small works for economy.

(e) Substitution of steel for cast-iron pipe in pumping stations, particularly in welded prefabricated headers. It is usually cheaper, saves space and, on the high-pressure side, is not subject to sudden cracking or failure. In large works the steel pipe can be cement-lined; elsewhere the pipe diameter and well thickness should be selected to provide ample margin for corrosion.

APPLICATION OF ULTRAVIOLET RAY STERILIZATION TO WATER TREATMENT

BY F. WELLINGTON GILCREAS AND LOUIS DE LALLA*

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The power of sunlight to destroy bacterial life has long been known and appreciated. Effective disinfection in air, on surfaces and in water has been accomplished by exposure to the direct rays of the sun. Sunlight is an important factor in the self-purification of water in streams and in impounding reservoirs, and its effect in destroying bacteria, particularly intestinal bacteria, has been reported upon many times. The ordinary rays of sunlight play little part in this bactericidal action; the results are caused by the ultraviolet rays. Unfortunately, ultraviolet rays have little penetrative power and are readily absorbed by particles of dust, opaque substances, water, or glass. Their bactericidal effect, however, is marked.

An early method for sterilizing water was the use of ultraviolet light, generated by mecury vapor lamps. These lamps were first studied in France and employed for sterilization of small quantities of drinking water, but no practical installations on public water supplies were reported. (1) When this type of treatment became known in the United States it was for a time used on a few small water supplies. The method proved unsatisfactory for application to public water supplies and was replaced by chlorination, a cheaper and much more easily controlled process for destroying bacteria in large volumes of water. Ultraviolet light continued to be employed for swimming pools and small, individual home supplies where cost was not a factor and where it was desired to avoid the unpleasant taste and odor then frequently associated with chlorination.

From the first experimental installations, it was readily determined that, to be effective, ultraviolet sterilization must be applied to clear water, which meant it could be used only when the water had been filtered. This introduced complications and also raised the cost.

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Because of their low penetrative power ultraviolet rays would not treat water at any great depth, and their bactericidal efficiency was seriously lessened by traces of turbidity, color, or iron, found even in filtered supplies (2). Mercury vapor lamps could not be placed in direct contact with the water, because of the high temperature at which they operated, so it was necessary to separate the water from the lamp by a quartz tube, which permitted passage of the ultraviolet rays but sharply limited their effective penetration into the water. Consequently, the ultraviolet ray method of water treatment gradually lost favor and practically disappeared as an acceptable disinfecting process, even in artificial bathing pools or individual water supplies.

Advances in the construction of ultraviolet lamps with the production of the "cold" lamp stimulated investigation of the application of this method of treatment to water. Because of the low temperature (70° to 80°F.) at which the newer germicidal lamps operate, they can be submerged in water without significant reduction in their operating efficiency. Thus, a more efficient installation was possible and a greater degree of penetration of the rays into the water could be obtained. Commercial equipment for treating water by passage over cold ultraviolet lamps has now been developed, particularly for use in swimming pools and small individual water supplies (3, 4).

The other limitation of the procedure, the absorption of ultraviolet light by substances present in the water, remains to be considered when judging the practical value of this treatment. A study of the bactericidal effectiveness of ultraviolet light as applied to water seemed desirable, in order to determine the limitations of such treatment and of the newly designed equipment employing it. A commercial ultraviolet sterilizer, developed for use in small water supplies, was obtained. It consists of six 30-watt ultraviolet lamps, each providing about 10 watts of ultraviolet light in the 2,537 Angstrom unit range, which is the most effective wavelength. The lamps are placed in series inside of metal tubes, highly polished on the inside, which serve as reflectors to increase the penetrative power of the ultraviolet light into the water. The water passes over the lamps in a spiral fashion and the depth of the water over the tube is approxi-

mately three-eighths of an inch. This is well within the usual penetrative power of ultraviolet light in clear water, where 10% penetration or 90% absorption would be obtained only at a distance possibly equivalent to several feet.

The capacity of the commercial device used in this investigation permits the treatment of 750 gallons of water per hour. The length of the tubes provides 15 feet of flow and thus a time of exposure of the water to the ultraviolet rays of approximately 4.2 seconds. The shallow depth of the water and the reflecting surface on the entire carrying pipe allow full penetration of the ultraviolet light in clear water. With the presumed spiral flow through the pipe, adequate stirring and mixing is possible permitting the light to reach the bacteria without complete blocking by small suspended particles of turbidity or iron in the water.

To study the effect of this equipment, Albany city tap water was used, the chemical analysis of which is shown in Table 1. This water as received in the laboratory had only a very low residual chlorine content (0.01 ppm), and to it was added a suspension of *Bacterium coli* in buffered dilution water. The culture of *Bact. coli* had been grown on solid medium, removed, and suspended in the buffered dilution water, so as to produce varying numbers of coliform bacteria per milliliter of suspension. The suspension was then added to the water to provide coliform densities ranging from a Most Probable Number (M.P.N.) of 2,400 to 240,000. The water was pumped through the ultraviolet ray sterilizer at the rated capacity of the device, 750 gallons per hour, or an exposure time of about 4.2 seconds. Samples of the effluent from the treatment were collected at the beginning, at the middle, and at the end of each run and were examined

TABLE 1.—CHEMICAL ANALYSIS OF TAP WATER, ALBANY, NEW YORK

	ppm		ppm		ppm
Color	5.0	Chloride	2.0	Free ammonia	0.002
Turbidity	0.1	Oxygen consumed	1.9	Albuminoid ammonia	0.048
Iron	0.03	Alkalinity	43.0	Nitrites	0.001
Manganese	0.02	Hardness	66.0	Nitrates	0.02
		pH	8.5		

to determine coliform density. The results indicate that, even with high concentrations of coliform microörganisms, complete destruction could be obtained in clear water within the time and under the conditions of treatment with this equipment.

To determine the effects of rates of flow higher than 750 gallons per hour, the number of lights was gradually reduced by cutting out first one, then two, and more lights. By decreasing the time of exposure to the ultraviolet light rays, the effect was the same as pumping water through the machine at more than the rated capacity. Table 2 shows that, even with an exposure time of only 2.8 seconds, effective destruction of coliform bacteria was obtained. Consequently, with rates of flow higher than specified, or with the failure of one or two lamps, reliable treatment could still be anticipated.

One of the problems that arises with the use of ultraviolet ray tubes is the deposition on the tubes and on the reflecting surfaces of deposits from the water. These gradually build up an opaque coating and reduce markedly the amount of ultraviolet light available for penetration into the water, with resulting loss of bactericidal effect. As the experimental machine was used, this effect became very apparent. As is shown in Table 3, when this coating had accumulated and the equipment was operating at its rated capacity of 750 gallons per hour, effective disinfection could not be obtained. The lights were removed and cleaned, as were the reflectors, and put back in service, when full efficiency was again secured.

As these first experiments were made only with water free from appreciable concentrations of color or turbidity, it seemed desirable to determine the efficiency of ultraviolet ray treatment of water when color, turbidity and a combination of both conditions were present.

A suspension of diatomaceous earth was added to Albany tap water to produce turbidities ranging from 10 to 125 ppm. Coliform bacteria were added to these turbid suspensions in the same manner as to the clear water and the resulting test portions were subjected to the action of ultraviolet light. The results are shown in Table 4 and suggest that with a turbidity of 125 ppm, effective bactericidal treatment will result when the six lamps are in operation. The effect of reducing the number of lamps and the time of exposure—that

TABLE 2.—EFFECT OF INCREASING COLIFORM DENSITY AND DECREASING TIME EXPOSURE TO ULTRAVIOLET LIGHTS

Time of exposure to lights (seconds)	Coliform bacteria—median M.P.N. per 100 ml		Coliform bacteria—treated water	
	Untreated water	Treated water	No. samples positive	No. samples negative
4.2	4,300	<3.6	0	30
4.2	33,500	<3.6	0	30
4.2	240,000	<3.6	0	15
3.5	4,300	<3.6	0	30
2.8	4,300	<3.6	0	15
2.1	4,300	3.6	10	5
1.4	6,800	335	12	0

TABLE 3.—ULTRAVIOLET RAY TREATMENT OF CLEAR WATER—REDUCTION IN DENSITY OF BACTERIA OF COLIFORM GROUP AND EFFECT OF DEPOSITS ON ULTRAVIOLET LAMPS

Time of exposure to lights (seconds)	Coliform bacteria—median M.P.N. per 100 ml		Coliform bacteria—treated water	
	Untreated water	Treated water	No. samples positive	No. samples negative
		No deposits on tubes		
4.2	7,500	<3.6	0	9
3.5	4,300	<3.6	0	30
2.8	4,300	<3.6	0	15
		Opaque deposits on tubes		
4.2	4,300	<3.6	8	7
3.5	2,400	9.1	23	4
2.8	4,300	8.2	14	4

TABLE 4.—ULTRAVIOLET TREATMENT OF WATER CONTAINING TURBIDITY, COLOR AND IRON

Exposure time (seconds)	Turbidity (ppm)	Color (ppm)	Iron (ppm)	Coliform bacteria—median M.P.N. per 100 ml		Coliform bacteria—treated water	
				Untreated water	Treated water	Samples positive	Samples negative
4.2	10			4,300	<3.6	0	15
4.2	25			4,300	<3.6	0	15
4.2	35			4,300	<3.6	1	14
4.2	100			4,300	<3.6	0	3
4.2	125			7,500	<3.6	0	9
3.5	20			4,300	<3.6	0	15
3.5	100			3,900	<3.6	2	13
2.8	50			6,400	<3.6	0	15
2.8	75			9,300	<3.6	2	10
2.8	100			4,300	3.6	8	7
4.2		15		4,300	<3.6	1	14
4.2		25		2,400	<3.6	0	15
4.2		75		3,350	<3.6	0	6
4.2		125		4,300	23.0	15	0
3.5		25		9,300	<3.6	0	15
3.5		75		4,300	<3.6	7	8
2.8		25		4,300	<3.6	2	13
2.8		75		9,300	15.0	3	0
4.2	3	30	1	4,300	<3.6	3	12
4.2	10	30	1	4,300	<3.6	2	13
4.2	10	90	3	4,300	<3.6	0	15
4.2	15	150	5	4,300	<3.6	0	15
4.2	20	200	6	1,665	<3.6	5	7
4.2	25	90	3	5,900	<3.6	0	12
4.2	85	90	3	2,400	<3.6	0	12
3.5	10	30	1	9,300	<3.6	2	13
3.5	20	200	6	2,400	23.0	15	0
3.5	75	90	3	9,300	<3.6	3	6
2.8	10	90	3	2,400	3.6	8	7
2.8	75	90	3	4,300	3.6	7	2

is, an action comparable to increasing the rate of flow—indicated the maximum turbidity that could be permitted while still securing adequate disinfection. With four lamps in operation, complete destruction of coliform bacteria was obtained only with turbidity less than 75 ppm.

To simulate color, an infusion of tea was added to Albany tap water, to give a range from 15 to 125 ppm, and the experiments were repeated. With full capacity of the equipment providing at least 4.2 seconds exposure, soluble color in the water did not interfere with the bactericidal effect of the ultraviolet rays until the color was greater than 75 ppm.

Since one of the earliest noted interferences with ultraviolet ray sterilization was iron in colloidal suspension, the effect of this condition was also studied by adding a solution of ferrous sulfate to Albany tap water, to provide concentrations of iron from 1 ppm to 6 ppm. Iron in colloidal suspension in water did not reduce the bactericidal efficiency of the ultraviolet light until the concentration was in excess of 5.0 ppm.

The adsorptive effect of organic matter on ultraviolet light is well known and might seriously interfere with treatment. The study was extended to include the effects of turbidity caused by organic material. Skim milk was added to Albany tap water to yield a turbidity of 30 ppm. Coliform bacteria were added as previously and results indicate that 30 ppm of this organic turbidity has the same effect as 100 ppm of inorganic turbidity. With turbidities due to organic matter much in excess of 30 ppm, effective disinfection at an exposure time of 4.2 seconds cannot be expected.

Each of the experiments described represented a separate investigation, but under ordinary operation conditions frequently turbidity, color and iron are present in the water at the same time. Thus, turbidity, color and iron were added to Albany tap water together, to determine the limit of the values which could be expected to produce no marked interference with ultraviolet ray activity in destroying coliform bacteria also added to the water. From this study, as shown in Table 4, it is apparent that with turbidities of 85 ppm, iron 3 ppm, and color (inorganic) 90 ppm, or with turbidities of 15 ppm, iron 5 ppm, and color (inorganic) 150 ppm,

effective destruction of test organisms can be anticipated at the rated capacity of the equipment.

Summary and Discussion

From the results of the investigations with one commercial device it is apparent that, when ultraviolet light is used as a disinfecting agent in water, effective destruction of coliform bacteria can be anticipated when either turbidity, color or iron is no greater than the limiting values determined in the individual experiments, and when the time of exposure to the light is at least 4.2 seconds and the depth of penetration required is no more than three-eighths of an inch.

All of these experiments were undertaken at a water temperature of approximately 5°C. or 41°F. When the operating temperature of ultraviolet lights is lowered much below that, a marked reduction in efficiency can be expected. When the temperature is less than 5°C. the ultraviolet ray output of the tube may drop to one-quarter or one-third of its normal capacity, with a corresponding decrease in the bactericidal efficiency of the equipment. The reduction in efficiency at low temperature, however, can be counterbalanced by operating the lamps at higher wattage. If they are operated at 36 watts, the efficiency at a lower temperature would be approximately that equivalent to operating at 30 watts at the higher water temperature.

The gradual deposition on the tubes of suspended matter, even from clear water, makes it essential that the tubes be kept clean and free from such deposit, if maximum bactericidal efficiency is to be obtained. Obviously, the greater the amount of organic matter, color and suspended material in the water treated, the more rapidly will interfering deposits accumulate on the tubes and reflectors, with subsequent marked reduction in bactericidal efficiency.

The major difficulty in ultraviolet light sterilization of water is the lack of a rapid field test to determine whether the water has been adequately treated. The only measure of treatment at present is a bacteriologic examination, which requires at least two days and does not furnish with sufficient rapidity data that can be used to control this type of disinfecting treatment. However, when properly applied, ultraviolet light will destroy coliform bacteria in water and may be relied upon for satisfactory disinfection. Much develop-

ment of commercial equipment for applying this treatment to water is necessary, in order that it may be fully effective and assure complete disinfection of the treated water at all times.

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THE USE OF CHEMICAL WEED KILLERS ON PUBLIC WATER-SUPPLY WATERSHED

BY DONALD W. LOISELLE*

[*Read at Chemists' Round Table, Feb. 19, 1953.*]

The Bridgeport Hydraulic Co. has been experimenting with chemical weed killers on its watershed since 1948. These experiments were started to determine the efficiency and economy of various sprays and to examine the results, with a view to reducing the amount of hand labor required on the watershed.

At present the flow lines of all reservoir and canals, as well as transmission line rights-of-way, are mowed at least once a year, preferably in the late summer. Machinery is used wherever possible in this work, but much of our shore line is rough and rocky and not accessible to mowing equipment. Therefore hand mowing is used for the majority of the work, but in the last few years labor has not been readily available for this work.

A survey of the whole field of chemical herbicides was made, to determine the best chemical to use from the standpoint of cost, safety, ease of application and growth-killing or retarding powers. We found that little or no work had been done with weed killers around or in a public water supply, so that we had to start from the beginning. E. I. duPont de Nemours & Co. offered to join us in this project and we accepted their offer.

DuPont laboratories and other agencies tested the different herbicides considered, to obtain the toxicological data. The chemicals were fed to rats and dogs, to determine the amount of chemical necessary for any noticeable change in diet and, if fed for a long period, the effect on the body organs, as well as the lethal doses. It was found that if these data could be applied to humans and the herbicides were used in accordance with recommended practice, there

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was absolutely no danger of chemical poisoning. The lethal dose for the average 200-lb man would be as follows:

245T	1.9 oz.
24D	2.2 oz.
Ammate	6.4 oz.
TCA	10.6 oz.
CMU	11.5 oz.

The aforementioned quantities apply to the pure salt; hence there is no chance in ordinary spraying operations that one could take in that amount of chemical.

The next consideration is what happens to the spray after it is applied to the weed. On this subject not too much is known, but what we in the water business are most interested in is what chemical products are finally formed and carried into the water supply. The 24D and 245T products and, to some extent, CMU form phenol as one of the breakdown products. As you know, phenol is one of our most potent enemies in water, because chlorine will bring out a definite phenol taste even in concentration of two parts per billion. Since it is possible under normal spraying that, if enough 24D or 245T were applied near a small reservoir, a rain might wash enough of the chemical into the water to give a taste, the 24D and 245T were not used in the experiments along water courses. However, small-scale experiments were run, away from the brooks.

The three chemicals used—ammate, TCA and CMU—were picked because they are all water-soluble or water-carried, as well as safe to use. The ammate that is sold on the market contains a chromate corrosion inhibitor. Since hexavalent chromium is one of the unwanted chemicals in a water supply, it was arranged to have ammate for our experiments without the chromate. As ammate is corrosive to metal, without the inhibitor precautions had to be taken to protect the sprayers. The sprayers using ammate were washed out with a strong alkali immediately after use and a mild alkali was left in the tanks until the next operation, or for at least one week.

TCA, which is the generic name for sodium tri-chloro-acetate, is not as corrosive as ammate, but ammate is safer to use. CMU is a trade name for 3-P (—chlorophenyl) —1, 1 dimethylurea. It is non-corrosive and is a wettable powder. It was developed primarily as a soil sterilizer, but subsequent experiments proved that it could be used as a selective herbicide.

We started a first field experiment in the late summer of 1949, after obtaining the favorable toxicological data. Ammate was first used because of its availability. Plots having an area of approximately 300 sq ft were laid out in a section of rip-rapping, near the Route 57 bridge crossing Saugatuck reservoir. The areas were sprayed with various dosages from ¼ to 1½ lb of ammate per gallon of water. We used a hand sprayer having a three-gallon capacity, so that we applied one gallon of spray per 100 sq ft of ground area. On an acre basis this would be 109 to 654 lb per acre, which is the standard basic term in most of the literature.

At the end of two weeks the results were not what we had hoped for, because the weed killing was spotty with no definite pattern. We consulted with the field representative of duPont about the results. Since ammate kills by leaf absorption and further transposal of the chemical down to the root system it was concluded that either the spray did not stay on the leaf long enough to be absorbed or since it was late in the season, the plants were more or less dormant and were not absorbing anything. We decided that in the future we would spray in the late spring, when the plants were having their maximum rate of growth. It was further decided, with all chemicals that use leaf absorption as their method of entrance, to add a spreader sticker chemical or wetting agent to the spray, to hold the herbicide in contact with the leaves. This was found later to be a "must" on plants having a waxy leaf.

Armed with the previous year's results, we started out in 1950 early in June to spray at two locations, with 16 plots in each location. The plots were not laid out in advance, but staked out as we sprayed. Three gallons of spray were used in each plot, all the weeds being sprayed thoroughly and as much ground as possible being covered.

Both ammate and TCA were used, both alone and in combinations. The following doses were used:

1¼ lb ammate per gallon of water
1 lb ammate per gallon of water
¾ lb ammate per gallon of water
½ lb ammate per gallon of water
¼ lb ammate per gallon of water
1¼ lb TCA per gallon of water
1 lb TCA per gallon of water
¾ lb TCA per gallon of water
½ lb TCA per gallon of water
¼ lb TCA per gallon of water
1 lb ammate plus ¼ lb TCA
¾ lb ammate plus ¼ lb TCA
½ lb ammate plus ¼ lb TCA
¼ lb ammate plus ¼ lb TCA
¾ lb ammate plus ¾ lb TCA
½ lb ammate plus ½ lb TCA

Spreader sticker was added at the rate of 4 oz per 100 gal of spray.

Three gallons of each of the above were sprayed on the plants, care being taken that all of the leaf area was wet, and the ground area covered varied from 456 to 756 sq ft, depending upon the amount of growth.

The first location sprayed was a section of the banks of the Means Brook Canal. The growth here was very heavy, containing several grasses, many of the common weeds and blackberry briers. The area sprayed extended along the canal from the water line back about 10 ft. The first plot was sprayed, then a space of 10 ft left as a control, and then the second plot sprayed.

The plots at this first location were observed at the end of the first week and then monthly for the rest of the year. The results at the end of the first week showed that the plots sprayed with ammate were progressing faster than those with TCA. The high dosages of ammate (1¼ and 1 lb per gal) showed almost complete kill at the end of the first week, except for some grass. At the end of the season the following summary of results was made:

Pounds of ammate and TCA per gal of water	Per cent of growth killed	Remarks
1¼ lb ammate	99	Small amount of grass left in spots
1 lb ammate	96	Some low-growing grass left
¾ lb ammate	94	Same as above
½ lb ammate	92	All tall growth killed. Some low grass and a blackberry bush showed life.
¼ lb ammate	85	All tall growth killed, except blackberry and elders. Most low grass survived.
1¼ lb TCA	98	100% kill of grass and sedge. Few elders left
1 lb TCA	98	Same as 1¼ lb TCA
¾ lb TCA	94	Almost as good as 1¼ and 1 lb TCA
½ lb TCA	94	98% of grass and most of the tall growth
¼ lb TCA	90	Grass kill almost equal to ½ lb TCA rate; some tendency for goldenrod and wild aster to survive.
1 lb ammate + ¼ lb TCA	98	Little life in some briers
¾ lb ammate + ¼ lb TCA	95	A little green in low grass; some briers have some life.
½ lb ammate + ¼ lb TCA	92	Some amount of low grass still green; some briers and elders show some life
¼ lb ammate + ¼ lb TCA	92	Some briers not entirely killed, but serious injury to all. Some low grass shows some life.
½ lb ammate + ½ lb TCA	99	Only a few briers show some life
¾ lb ammate + ¾ lb TCA	100	No green on any plant

The second location was a low flat area at the north end of Trap Falls Reservoir, which is partly under water when the reservoir is full, and the reservoir was full at the time of the experiments. The area was sprayed with the same dosages and combinations as the first location and in the same manner—namely, to wet completely

the leaf area of all plants. The plots, which were laid out as we sprayed, were slightly larger than the first location, because the plant growth was not as heavy. Some of the sprayed area was actually in the water. All the underwater growth which had leaves above the water surface was sprayed.

In the second area nearly every plot had 100% kill, because none of the elder and blackberry bushes, which were troublesome in the first location, were present in the second. The part which I believe is especially interesting to us is that all of the water weeds were killed and, even after the reservoir went down later in the summer, no weed growth came up.

In the fall of 1950 we computed the probable spraying costs for each plot. We then compared this cost against the results obtained and found that the TCA cost approximately 2½ to 3 times the ammate, to accomplish the same results. Hence we based our 1951 experimental spraying program more in the direction of ammate.

Also in 1950 we did a considerable amount of poison-ivy eradication spraying. Prior to 1950 we had a lot of man-hours lost because of poison-ivy allergy and we used to supply between 4 and 6 gal of calomel solution each year to our watershed and pipe-line workers. Therefore, in 1950 we sprayed one section of our orchard and also sprayed about one week ahead of our construction gang on a large pipe-line job. A dose of ¾ lb of ammate per gallon of water plus 4 oz of spreader sticker per 100 gal of water was used in the orchard and 1½ lb of ammate per gallon on the pipe-line. The heavier dose on pipe-lines was used because immediate kill was required and hand spray equipment was used, whereas in the orchard power sprayers were used. The results were excellent in both cases. The number of poison-ivy cases on the pipe gang started down immediately and for the rest of the year that gang had practically no lost time from the allergy. The amount of the poison-ivy solution (calomel) dropped to less than one gallon. The orchard results were similar during the picking season. Those working the sprayed area had no trouble, whereas, when the non-sprayed area was picked, we had many cases of poison ivy. We have since nearly completed the spraying of the whole orchard. However, it was subsequently found that dosages

of less than ¾ lb were not economical, because we got some re-growth the following year.

In 1951 we duplicated the experiments of 1950 with experimental plots in two new locations, both of which contained some underwater growth. The results were similar to the 1950 results. The original 1950 plots were kept under surveillance as to amount and kind of growth.

At first location along the canal the sprayed plots showed a very definite change in the type of growth. All the plots grew grass, but the ammate plots showed no weeds or briers. The plots showed some retardation in the growth rate, but the most startling thing was the complete eradication of the weeds.

At the second location a very definite retarding effect was noted in 1951. The parts of the plots, sprayed with ammate, which were under water, showed no growth, except some very short grass which, even in the fall, was only 2 to 3 in. in height. The part not under water showed the almost complete absence of weeds; only the grass remained and that was not heavy.

In 1952 the original 1950 plots were re-examined and at the first location several weeds were found growing in the sprayed area, which were re-seeded from the adjacent control or non-sprayed area. However, at the second location definite evidence of the spraying was noted, especially in the underwater areas. The grass in all the plots sprayed with the higher doses of ammate showed few weeds and the total growth was not as high as the control areas. Much of the underwater area was still clear, except for the low grass. These plots were not mowed this year, so that observations can be made again in 1953, which will be four years of control.

Early in 1952 duPont made available to us a new product, which they have since put on the market, called CMU. It was originally developed as a soil sterilizer, but we set up a series of experiments to test it for growth retardant as well as a soil sterilizer. Uses as a sterilizer on the watershed are numerous, such as in raceways, walks, driveways, rip-rapping and fire trails. Three experimental areas were picked for the soil-sterilizer experiment. The first was a newly cut fire trail. This trail was cut in February, 1952, through second-growth timber, 10 ft wide with overhanging trees, so that no direct

sunlight was present. The second location was the Popp's Mt. Tunnel driveway, which is in the open with no shade. This drive grows up very fast, requiring two or three mowings per year. The third location was the Hemlocks Reservoir raceway, in which brush grows up between the rocks and causes leaves and branches to pile up when water overflows the spillway.

All three locations were laid out in 0.01-acre plots, five to each location. This was done because CMU is sprayed on the ground, not on the foliage, and is absorbed by the roots directly. The dosages used were 20, 30, 40, 50 and 60 lb per acre. The amount of water used to carry the chemical is not of great importance, as long as its enough to distribute the chemical evenly over the entire plot.

The results were excellent. The plots in the open sunlight showed 100% kill in two to three weeks, but those in the shade took 6 to 7 weeks to reach the same 100% mark. In the first or shady location, the 20- and 30-lb dose plots showed that some sassafras bushes survived at the end of the summer, but with the richer doses nothing survived. Next summer will tell whether we get any re-growth, but at present the results look good. In other places that CMU has been used, after three years no growth has come back into the sprayed area, but in a few other locations some re-growth was observed in the second year. Some of the factors that determine how long CMU will remain effective are dosage, soil types, rainfall and plant species present. Thus as we learn more about this chemical, we have every reason to believe that long-lasting results can be obtained in most cases.

The use of CMU as a growth retardant is something we started because in some locations some growth is necessary to stabilize the soil, so as to prevent erosion. Two locations were picked: one the Trout Brook Valley siphon right-of-way, which is a ten-foot mowed strip with no shade; the second location, part of Eastern Lake transmission main right-of-way, which is in the woods, so that the grass receives no sunlight. These locations require some grass growth, to prevent erosion, but mowing must be done by hand. Each area was divided into five plots, each containing 0.01 acre, and the plots were sprayed with doses of 2, 4, 6, 8 and 10 lb per acre.

The plots were examined twice last summer and fall. At the first location some growth retarding was noted, even in the two-pound plot. The control plot had grass 14 to 24 in. high, while in the 2-, 4- and 6-lb plots, the grass grew only 4 to 8 in. high. In the 8- and 10-lb plots, some plants were killed, but the remaining were only 2 to 4 in. in height. In the second location, which was shady, the effect was slower, but the 10-lb plot showed good control, comparable to the 6-lb plot at the first location. Even in the two-pound plot, grass grew only 8 in., compared to the control growth of 16 in.

Both locations show great promise and, since neither the control nor the sprayed plots were mowed this year, we will see the effect next year.

Our experiments with 24D and 245T away from the brooks showed that ammate can be used in most cases in weed control just as easily, almost as cheaply and with much less danger of damage to nearby crops and shrubs.

The conclusions that we can draw after three years of experimenting are:

1. Ammate and CMU look like the two best herbicides to use on the watershed.
2. Ammate has been very good in the control of water-line growth.
3. Ammate can be used easily, economically and with no danger to the water supply.
4. If poison ivy is a problem to workers, it can be completely eradicated by ammate.
5. If CMU continues to retard growth in the next two years as it has the first year, it will go a long way toward solving the mowing of the different areas.
6. CMU in larger doses can be used to remove and stop re-growth of plants in areas where no growth is wanted.
7. CMU is at present high-priced, but the small amount used brings the cost per acre down to the level of other soil sterilizers, which are not safe to use around a water supply.

I feel that, from our past experiments and some future planned ones, our knowledge of chemical herbicides will be complete enough to warrant wholesale use of these chemicals to replace some of our hand mowing, with a considerable saving in time and expense.

PROCEEDINGS

JANUARY 1953 MEETING

HOTEL STATLER, BOSTON, MASS.

THURSDAY, JANUARY 22, 1953

President Stanley M. Dore in the Chair.

Secretary Knox announced the election of the following to membership in the Association:

Albert E. Mountain, Superintendent, Water Company, Westford, Mass.; Ronald I. Marshall, Assistant Superintendent, Water Department, Georgetown, Mass.; Robert Lee McGrath, Superintendent, Water Works, Pepperell, Mass.

Corporate:

Abington Water Works, Abington, Mass.; Board of Water Supply, Honolulu, Hawaii.

A paper on "Chlorination—A Ten-Year Review" was read by Attmore E. Griffin, Technical Service Director, Wallace & Tiernan Co., Inc., Newark, N. J.

A paper on "Operation of the Lynnfield, Massachusetts, Water Supply" was read by Clifton E. Hodgdon, Superintendent, Water District, Lynnfield Center, Mass.

A paper on "Corrosion Control at Danbury, Connecticut" was read by Ellis A. Tarlton, Consulting Engineer, Danbury, Conn.

February 1953 Meeting

Hotel Statler, Boston, Mass.

Thursday, February 19, 1953

President Stanley M. Dore in the Chair.

Secretary Knox announced the election of the following to membership in the Association:

Samuel L. Satcher, Superintendent, Auburn Water District, Auburn, Mass.; Ernest R. Coutermarsh, Chairman, Water Commission, Lebanon, N. H.; Kenneth P. Currie, District Manager, Caribou Water Works Corp., Mars Hill, Me.; Paul J. Higgins, Superintendent, Water Department, Medway, Mass.; Ernest Boyd Livesay, Superintendent, Brunswick-Topsham Water District, Brunswick, Me.; Anthony J. Molinaro, Superintendent, Water and Sewer Department, Franklin, Mass.; Eugene F. Redlon, Superintendent, Duxbury Fire and Water District, Duxbury, Mass.; Edgar P. Snow, President, C. W. Riva Company, Construction Engineers, Providence, R. I.; George H. Wile, Superintendent, Lisbon Water Department, Lisbon Falls, Me.

Associate:

M. C. Britt Co., Groton, Mass.

A paper on "Operation of the Waltham Water Department" was read by Clarence B. French, Civil Engineer, Water Division, Waltham, Mass.

A paper on "Washing Slow-Sand Filters with a Lateral Flow of Water" was read by Alexander J. Minkus, Engineer of Purification, Metropolitan District Water Bureau, Hartford, Conn.

A film, "Deep Waters", was presented by the Layne-New York Co., Inc.

MARCH 1953 MEETING

HOTEL CONTINENTAL, CAMBRIDGE, MASS.

THURSDAY, MARCH 19, 1953

President Stanley M. Dore in the Chair.

Secretary Knox announced the election of the following to membership in the Association:

Carl A. Arenander, Partner, Malcolm Pirnie Engineers, New York, N. Y.; Frederick W. Anthony, General Foreman, Water Department, New Bedford, Mass.; John H. Miller, Superintendent, Water Department, Warwick, R. I.; Robert D. Mitchell, Consulting Engineer, Malcolm Pirnie Engineers, New York, N. Y.; Frank X. Pooley, Superintendent, Water Department, Woonsocket, R. I.; John W. Whitehead, Superintendent, Water Department, Dartmouth, Mass.

Upon motion duly made and seconded, it was VOTED to appoint the following as the Nominating Committee: Sidney S. Anthony, Arthur C. King, Donald C. Calderwood, Arthur L. Shaw and Harold W. Griswold.

An official welcome was extended to the Association by John J. Curry, City Manager of Cambridge.

A paper, "Responsibilities and Aims of the Water Works Industry," was read by Charles H. Capen, President, American Water Works Association, Wanaque, N. J.

A paper on "Operation of Plant Facilities" was read by William H. McGuinness, Superintendent, Water Department, Cambridge, Mass.

A paper on "Improvements to Pumping and Filtration Facilities at Cambridge Filter Plant" was read by Thomas R. Camp, Consulting Engineer, Boston, Mass.

A paper on "Planning of a New Water-Works Maintenance Plant" was read by Theodore G. Ames, Architect, Boston, Mass.

AWWA C100-52T
(*Formerly* 7C.1)

Tentative

Standard Specifications

for

CAST-IRON PRESSURE FITTINGS

American Water Works Association

New England Water Works Association

This document presents in slightly revised form the "special castings" portion of "Standard Specifications for Cast-Iron Water Pipes and Special Castings—7C.1–1908" which was adopted by AWWA on May 12, 1908. Although it has been superseded for new installations by various American Standards for cast-iron pipe and fittings (ASA specifications A21.2, A21.6, A21.8, and A21.10), it is still valid and useful as a basis for replacing fittings in distribution systems constructed according to the original 1908 specifications.

Certain revisions of the 1908 text approved by AWWA on Dec. 31, 1952, and by NEWWA on Jan. 22, 1953, are contained in the present document.

AMERICAN WATER WORKS ASSOCIATION
Incorporated
521 Fifth Avenue, New York 17, N.Y.

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Latest Revisions to C100 (Formerly 7C.1–1908)

Specifications C100 were revised Dec. 31, 1952. The changes consist of:

1. Removal of the material on cast-iron pipe.
2. Revision of the tables of weights for cast-iron fittings.
3. Elimination of the old Type 1 reducer and Type 1 Y branch.
4. Changes in bell and spigot dimensions (Table 1).
5. Changes in offset design to give three offsets per pipe size instead of one (Table 4).
6. Larger dimensions for blowoff branches with manhole (Table 8) and for manhole pipe (Table 9).
7. Simplification and correction of illustrations, particularly Fig. 1, 6, 11, 12, and 13.
8. Miscellaneous additions (such as that of bell-and-bell combinations) and corrections.

These revisions have been incorporated into the text of the Second Edition, first published in March 1953. The text of the earlier edition thus becomes obsolete.

This printing contains all the revisions and changes summarized above.

Tentative Standard Specifications for

Cast-Iron Pressure Fittings

Sec. 1—Description of Fittings

The fittings shall be made with bell and spigot joints and shall conform, within the specified tolerances, to the dimensions given in the tables forming a part of these specifications.

For pipe 4–12 in., one class of fittings shall be furnished, made from Class D patterns. Spigot ends shall have reduced outside diameters as shown by Table 1 and shall taper back for a distance of 6 in.

For pipe 14–24 in., two classes of fittings shall be furnished—Class B, and Class D, on which the letters "B" and "D", respectively, shall be cast. For pipe 30–60 in., four classes of fittings—Classes A, B, C, and D, shall be furnished and they shall have cast on them the letter of the class to which they belong.

Classes A, B, C, and D fittings provide for the following working heads and pressures:

Class	Head	Pressure
	ft	*psi*
A	100	43
B	200	86
C	300	130
D	400	173

The flanges on all manhole castings and manhole covers shall be faced and drilled as shown in the tables. The manufacturer shall furnish mild steel bolts with square heads, hexagon nuts and gaskets.

Sec. 2—Variation in Diameter

Sockets and spigots shall be tested with circular gages. Tolerances of fittings made from standard patterns are:

Size Range	Tolerance
in.	*in.*
4–16	± 0.12
18–24	± 0.15
30–42	± 0.20
48–60	± 0.24

Sec. 3—Variation in Thickness

The thickness tolerance for fittings with standard wall thickness of less than 1 in. shall be minus 0.12 in. The tolerance for fittings with standard wall thickness 1 in. and more shall be minus 0.15 in. An additional tolerance of 0.03 in. is permitted for spaces not exceeding 8 in. in length in any direction.

Sec. 4—Variation in Weight

The weight tolerance for 4–12 in. standard fittings is ± 10 per cent. For 14–60 in. fittings, the tolerance is ± 8 per cent. For bends and wyes, the tolerance is ± 12 per cent.

No weight shall be paid for that is in excess of the amount allowed by maximum tolerance. No fitting shall be accepted that weighs less than the amount allowed by minimum tolerance.

Sec. 5—Marking

Each fitting shall have cast, on the outside, the initials of the maker's name

and the class. As many as four special initials and the year may also be cast when required by the customer. The weight shall be painted conspicuously on each fitting.

Sec. 6—Quality of Iron

All fittings shall be made of cast iron of good quality. The metal of the castings shall be strong, tough, of even grain, and soft enough to drill and cut satisfactorily. The manufacturer shall have the right to make and break three bars from each heat and report the average results of the three tests.

Sec. 7—Tests of Material

At least one test bar of the metal used, 26 in. long by 2 in. wide and 1 in. thick, shall be made and tested from each heat. The bars, when placed flatwise upon supports 24 in. apart, and loaded in the center, shall support a load of 2,000 lb and show a deflection of not less than 0.30 in. before breaking. If preferred, tensile bars which will show a breaking point of not less than 20,000 psi shall be made.

Sec. 8—Quality of Castings

The fittings shall be smooth and free from defects of every nature which would make them unfit for the use for which they are intended. No plugging or filling will be allowed.

Sec 9—Cleaning and Inspection

All fittings shall be thoroughly cleaned and subjected to a careful inspection.

Sec. 10—Linings and Exterior Coatings

Any particular lining or coating which is to be applied to the fittings shall be specified in the agreement made at the time of purchase. Separate specifications for cement-mortar lining (ASA A21.4; also known as AWWA C104) have been provided as a part of specifications for pipe.

No fittings for water works service shall be furnished without protective coating unless specifically ordered by the purchaser.

Sec. 11—Weighing

The fittings shall be weighed under the supervision of the engineer before the application of any lining or coating other than hot or cold bituminous dip or paint. If desired by the engineer, the fittings shall be weighed after their delivery, and the weights so ascertained shall be used in the final settlement, provided such weighing is done by a legalized weighmaster. Bids shall be submitted and a final settlement made upon the basis of a ton of 2,000 lb.

Sec. 12—Men and Material

The manufacturer shall provide all tools, testing machines, and labor necessary for the required testing, inspection and weighing at the foundry. If specified on the order, the manufacturer shall furnish the test results and a sworn statement that all of the tests have been made as specified.

Sec. 13—Power to Inspect

The engineer shall be at liberty at all times to inspect the material at the foundry, and the molding, casting, and coating of the fittings. All castings shall be subject to his inspection and approval, and he may reject any casting which is not in conformity with the specifications or drawings.

Sec. 14—Inspector to Report

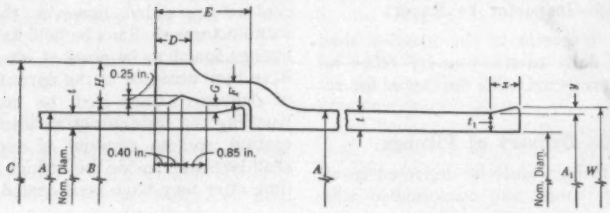
The inspector at the foundry shall report daily to the foundry office all fittings rejected, with the causes for rejection.

Sec. 15—Delivery of Fittings

All fittings must be delivered in all respects sound and conformable with these specifications. The inspection shall not relieve the manufacturer of his obligations under this section, and any defective fittings which may have passed the engineer at the works or elsewhere shall be at all times liable to rejection when discovered, until the final completion and adjustment of the contract; provided, however, that the manufacturer shall not be held liable for fittings found to be cracked after they have been accepted at the agreed point of delivery. Care shall be taken in handling the fittings not to injure the coating, and no material of any kind shall be placed inside the fittings at any time after they have been coated.

Sec. 16—Definition of "Engineer"

Wherever the word "engineer" is used herein it shall be understood to refer to the engineer or inspector acting for the purchaser and to his properly authorized agents, limited by the particular duties entrusted to them.



3-in. and 14- to 60-in. Spigots

4- to 12-in. Spigots

x and *y* Common to All Sizes

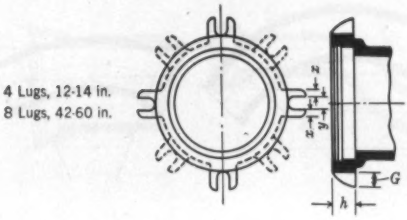
$x = \frac{3}{4}$ in. on 4-6 in.
$y = \frac{3}{16}$ in. on 4-12 in.

$x = 1$ in. on 8-60 in.
$y = \frac{1}{4}$ in. on 14-60 in.

TABLE 1. Standard Dimensions of Bells and Spigots for Fittings

Size *in.*	Class	Dimensions—*in.*											
		A	A_1*	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*t*	t_1*	*W*
3	D	3.96		4.66	7.26	1.25	3.50	0.65	0.35	1.30	0.48		4.34
4	D	5.00	4.90	5.70	8.30	1.50	4.00	0.65	0.40	1.30	0.52	0.47	5.28
6	D	7.10	7.00	7.80	10.60	1.50	4.00	0.70	0.40	1.40	0.55	0.50	7.38
8	D	9.30	9.18	10.00	13.00	1.50	4.00	0.75	0.41	1.50	0.60	0.54	9.56
10	D	11.40	11.25	12.10	15.30	1.50	4.00	0.80	0.42	1.60	0.68	0.60	11.63
12	D	13.50	13.35	14.20	17.60	1.50	4.00	0.85	0.42	1.70	0.75	0.68	13.73
14	B	15.30		16.10	19.50	1.50	4.00	0.85	0.40	1.70	0.66		15.80
14	D	15.65		16.45	20.05	1.50	4.00	0.90	0.40	1.80	0.82		16.15
16	B	17.40		18.40	22.00	1.75	4.00	0.90	0.50	1.80	0.70		17.90
16	D	17.80		18.80	22.60	1.75	4.00	1.00	0.50	1.90	0.89		18.30
18	B	19.50		20.50	24.30	1.75	4.00	0.95	0.50	1.90	0.75		20.00
18	D	19.92		20.92	25.12	1.75	4.00	1.05	0.50	2.10	0.96		20.42
20	B	21.60		22.60	26.60	1.75	4.00	1.00	0.50	2.00	0.80		22.10
20	D	22.06		23.06	27.66	1.75	4.00	1.15	0.50	2.30	1.03		22.56
24	B	25.80		26.80	31.00	2.00	4.00	1.05	0.50	2.10	0.89		26.30
24	D	26.32		27.32	32.32	2.00	4.00	1.25	0.50	2.50	1.16		26.82
30	A	31.74		32.74	37.34	2.00	4.50	1.15	0.50	2.30	0.88		32.24
30	B	32.00		33.00	37.60	2.00	4.50	1.15	0.50	2.30	1.03		32.50
30	C	32.40		33.40	38.60	2.00	4.50	1.32	0.50	2.60	1.20		32.90
30	D	32.74		33.74	39.74	2.00	4.50	1.50	0.50	3.00	1.37		33.24
36	A	37.96		38.96	43.96	2.00	4.50	1.25	0.50	2.50	0.99		38.46
36	B	38.30		39.30	44.90	2.00	4.50	1.40	0.50	2.80	1.15		38.80
36	C	38.70		39.70	45.90	2.00	4.50	1.60	0.50	3.10	1.36		39.20
36	D	39.16		40.16	46.96	2.00	4.50	1.80	0.50	3.40	1.58		39.66
42	A	44.20		45.20	50.80	2.00	5.00	1.40	0.50	2.80	1.10		44.70
42	B	44.50		45.50	51.50	2.00	5.00	1.50	0.50	3.00	1.28		45.00
42	C	45.10		46.10	52.90	2.00	5.00	1.75	0.50	3.40	1.54		45.60
42	D	45.58		46.58	54.18	2.00	5.00	1.95	0.50	3.80	1.78		46.08
48	A	50.50		51.50	57.50	2.00	5.00	1.50	0.50	3.00	1.26		51.00
48	B	50.80		51.80	58.40	2.00	5.00	1.65	0.50	3.30	1.42		51.30
48	C	51.40		52.40	60.00	2.00	5.00	1.95	0.50	3.80	1.71		51.90
48	D	51.98		52.98	61.38	2.00	5.00	2.20	0.50	4.20	1.96		52.48
54	A	56.66		57.66	64.06	2.25	5.50	1.60	0.50	3.20	1.35		57.16
54	B	57.10		58.10	65.30	2.25	5.50	1.80	0.50	3.60	1.55		57.60
54	C	57.80		58.80	66.80	2.25	5.50	2.15	0.50	4.00	1.90		58.30
54	D	58.40		59.40	68.20	2.25	5.50	2.45	0.50	4.40	2.23		58.90
60	A	62.80		63.80	70.60	2.25	5.50	1.70	0.50	3.40	1.39		63.30
60	B	63.40		64.40	71.80	2.25	5.50	1.90	0.50	3.70	1.67		63.90
60	C	64.20		65.20	73.60	2.25	5.50	2.25	0.50	4.20	2.00		64.70
60	D	64.82		65.82	75.22	2.25	5.50	2.60	0.50	4.70	2.38		65.32

* For sizes 3 in. and 14–50 in., $A_1 = A$ and $t_1 = t$.



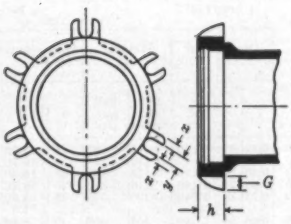


TABLE 2. Standard Lugs

Nominal Diameter Outlet *in.*	Number of Lugs	Approx. Weight, Bell Lugs *lb*	Approx. Weight, Spigot Lugs *lb*	Nominal Diameter Outlet *in.*	Number of Lugs	Approx. Weight, Bell Lugs *lb*	Approx. Weight, Spigot Lugs *lb*
12	4	30	45	30	6	80	125
14	4	30	45	36	6	85	135
16	6	45	70	42	8	130	200
18	6	50	75	48	8	130	210
20	6	50	75	54	8	145	230
24	6	50	80	60	8	150	250

Two pairs of lugs are placed on the vertical axis of each bell, the others at equal distances around circumference. As there are only six lugs on fittings in the 16–36-in. size range, purchasers should furnish a sketch showing location of lugs. On spigot end of pipe, unless otherwise specified, lug faces are located 21 in. from end of spigot. This dimension varies for fittings. *h* is equal depth of bell on all sizes.

G equals 2.50 in., *x* equals 1.25 in., *y* equals 1.63 in. for 12 to 24 in. inclusive.

G equals 3.00 in., *x* equals 1.50 in., *y* equals 2.00 in. for 30 to 60 in. inclusive.

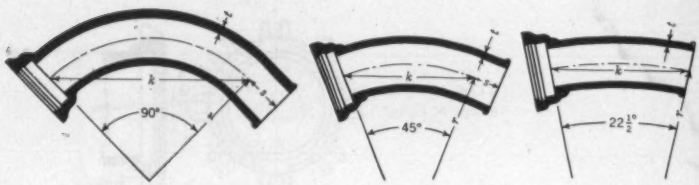


TABLE 3. Standard Bends

¼ Bends (90°)						⅛ Bends (45°)						1/16 Bends (22½°)			
Size *in.*	Class	Dimensions* *in.*			Weight† *lb*		Size *in.*	Class	Dimensions* *in.*			Weight† *lb*		Dimensions* *in.*		Weight† *lb*	
		l	*r*	*k*	Bell and Spigot	Bell and Bell			*t*	*r*	*k*	Bell and Spigot	Bell and Bell	*r*	*k*	Bell and Spigot	Bell and Bell
4	D	0.52	16	22.60	90	100	4	D	0.52	24	18.40	75	90	48	18.70	75	90
							6	D	0.55	24	18.40	110	125	48	18.70	110	125
6	D	0.55	16	22.60	135	145	8	D	0.60	24	18.40	155	180	48	18.70	155	180
							10	D	0.68	24	18.40	210	240	48	18.70	210	240
8	D	0.60	16	22.60	200	200	12	D	0.75	24	18.40	270	305	48	18.70	270	305
10	D	0.68	16	22.60	285	275	14	B	0.66	36	27.60	365	400	72	28.10	315	400
							14	D	0.82	36	27.60	440	475	72	28.10	380	475
12	D	0.75	16	22.60	365	350	16	B	0.70	36	27.60	445	500	72	28.10	385	500
							16	D	0.89	36	27.60	550	605	72	28.10	480	605
14	B	0.66	18	25.50	410	400	18	B	0.75	36	27.60	555	590				
							18	D	0.96	36	27.60	665	730	72	28.10	460	590
14	D	0.82	18	25.50	495	470								72	28.10	580	730
							20	B	0.80	48	36.70	755	825	96	37.50	670	825
16	B	0.70	24	34.00	590	590	20	D	1.03	48	36.70	965	1,045	96	37.50	860	1,045
							24	B	0.89	60	45.90	1,165	1,255	120	46.80	1,055	1,255
16	D	0.89	24	34.00	740	720	24	D	1.16	60	45.90	1,520	1,625	120	46.80	1,375	1,625
18	B	0.75	24	34.00	705	700	30	A	0.88	60	45.90	1,480	1,645	120	46.80	1,345	1,645
							30	B	1.03	60	45.90	1,690	1,830	120	46.80	1,530	1,830
18	D	0.96	24	34.00	895	870	30	C	1.20	60	45.90	1,980	2,145	120	46.80	1,795	2,145
							30	D	1.37	60	45.90	2,285	2,490	120	46.80	2,075	2,490
20	B	0.80	24	34.00	835	825	36	A	0.99	90	68.90	2,500	2,885	180	70.20	2,500	2,885
							36	B	1.15	90	68.90	2,920	3,360	180	70.20	2,920	3,360
20	D	1.03	24	34.00	1,070	1,045	36	C	1.36	90	68.90	3,450	3,960	180	70.20	3,450	3,960
							36	D	1.58	90	68.90	4,020	4,605	180	70.20	4,020	4,605
24	B	0.89	30	42.40	1,275	1,255											
							42	A	1.10	90	68.90	3,280	3,820	180	70.20	3,280	3,820
24	D	1.16	30	42.40	1,665	1,625	42	B	1.28	90	68.90	3,785	4,370	180	70.20	3,785	4,370
							42	C	1.54	90	68.90	4,580	5,280	180	70.20	4,580	5,280
30	A	0.88	36	50.90	1,820	1,850	42	D	1.78	90	68.90	5,315	6,115	180	70.20	5,315	6,115
30	B	1.03	36	50.90	2,090	2,075	48	A	1.26	90	68.90	4,250	4,905	180	70.20	4,250	4,905
							48	B	1.42	90	68.90	4,800	5,545	180	70.20	4,800	5,545
30	C	1.20	36	50.90	2,450	2,430	48	C	1.71	90	68.90	5,815	6,710	180	70.20	5,815	6,710
							48	D	1.96	90	68.90	6,715	7,755	180	70.20	6,715	7,755
30	D	1.37	36	50.90	2,825	2,820											
							54	A	1.35	90	68.90	5,180	6,025	180	70.20	5,180	6,025
36	A	0.99	48	67.90	3,000	3,025	54	B	1.55	90	68.90	5,975	6,960	180	70.20	5,975	6,960
							54	C	1.90	90	68.90	7,330	8,510	180	70.20	7,330	8,510
36	B	1.15	48	67.90	3,500	3,525	54	D	2.23	90	68.90	8,635	10,020	180	70.20	8,635	10,020
							60	A	1.39	90	68.90	5,960	6,975	180	70.20	5,960	6,975
36	C	1.36	48	67.90	4,145	4,155	60	B	1.67	90	68.90	7,110	8,245	180	70.20	7,110	8,245
							60	C	2.00	90	68.90	8,585	9,970	180	70.20	8,585	9,970
36	D	1.58	48	67.90	4,830	4,835	60	D	2.38	90	68.90	10,230	11,870	180	70.20	10,230	11,870

* For ¼ bends, S in the 4- and 6-in. sizes equals 8 in.; in the 8-in. size, S = 10 in.; and in the 10–36-in. size range, S = 12 in. For ⅛ bends, S = 6 in. in the 4–30-in. sizes. For 1/16 bends, S = 6 in. in the 4–12-in. sizes.
† All weights are rounded out to the nearest 5-lb multiple.

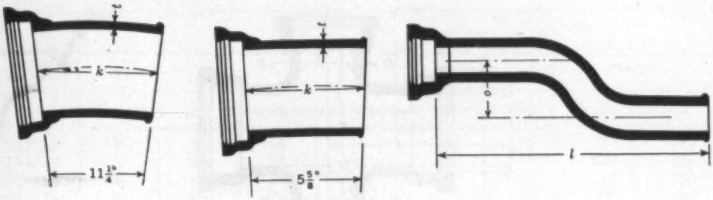


TABLE 4. Standard Bends and Offsets

Size *in.*	Class	11¼° Bends — Dimensions *in.*			11¼° Bends — Weight* *lb*		5⅝° Bends — Dimensions *in.*		5⅝° Bends — Weight* *lb*		Size *in.*	Class	Offsets — Dimensions *in.*		Offsets — Weight *lb*
		t	*r*	*k*	Bell and Spigot	Bell and Bell	*r*	*k*	Bell and Spigot	Bell and Bell			*O*	*L*	
4	D	0.52	120	23.52	70	95					4	D	6	27	80
6	D	0.55	120	23.52	105	140					4	D	12	30	90
8	D	0.60	120	23.52	150	200					4	D	18	38	110
10	D	0.68	120	23.52	205	265									
12	D	0.75	120	23.52	260	340					6	D	6	28	120
											6	D	12	34	145
14	B	0.66	180	35.28	375	455					6	D	18	41	175
14	D	0.82	180	35.28	450	545									
16	B	0.70	180	35.28	455	565					8	D	6	29	180
16	D	0.89	180	35.28	565	695					8	D	12	36	220
18	B	0.75	180	35.28	540	670					8	D	18	43	260
18	D	0.96	180	35.28	685	835									
											10	D	6	30	245
20	B	0.80	240	47.05	800	955	480	47.10	800	955	10	D	12	38	305
20	D	1.03	240	47.05	1,030	1,215	480	47.10	1,030	1,215	10	D	18	46	365
24	B	0.89	240	47.05	1,060	1,255	480	47.10	1,060	1,255					
24	D	1.16	240	47.05	1,375	1,620	480	47.10	1,375	1,620	12	D	6	34	350
											12	D	12	45	450
30	A	0.88	240	47.05	1,345	1,640	480	47.10	1,345	1,640	12	D	18	56	550
30	B	1.03	240	47.05	1,530	1,830	480	47.10	1,530	1,830					
30	C	1.20	240	47.05	1,795	2,145	480	47.10	1,795	2,145	14	B	6	35	375
30	D	1.37	240	47.05	2,075	2,490	480	47.10	2,075	2,490	14	B	12	46	475
											14	B	18	57	580
36	A	0.99	240	47.05	1,795	2,180	480	47.10	1,795	2,180					
36	B	1.15	240	47.05	2,095	2,540	480	47.10	2,095	2,540	14	D	6	35	455
36	C	1.36	240	47.05	2,470	2,980	480	47.10	2,470	2,980	14	D	12	46	580
36	D	1.58	240	47.05	2,875	3,460	480	47.10	2,875	3,460	14	D	18	57	710
42	A	1.10	240	47.05	2,370	2,905	480	47.10	2,370	2,905	16	B	6	35	460
42	B	1.28	240	47.05	2,720	3,305	480	47.10	2,720	3,305	16	B	12	48	600
42	C	1.54	240	47.05	3,290	3,985	480	47.10	3,290	3,985	16	B	18	58	710
42	D	1.78	240	47.05	3,815	4,615	480	47.10	3,815	4,615					
											16	D	6	35	570
48	A	1.26	240	47.05	3,055	3,715	480	47.10	3,055	3,715	16	D	12	48	750
48	B	1.42	240	47.05	3,455	4,195	480	47.10	3,450	4,195	16	D	18	58	895
48	C	1.71	240	47.05	4,180	5,075	480	47.10	4,180	5,075					
48	D	1.96	240	47.05	4,825	5,870	480	47.10	4,825	5,870					
54	A	1.35	240	47.05	3,740	4,585	480	47.10	3,740	4,585					
54	B	1.55	240	47.05	4,315	5,305	480	47.10	4,315	5,305					
54	C	1.90	240	47.05	5,285	6,470	480	47.10	5,285	6,470					
54	D	2.23	240	47.05	6,225	7,610	480	47.10	6,225	7,610					
60	A	1.39	240	47.05	4,320	5,330	480	47.10	4,320	5,330					
60	B	1.67	240	47.05	5,125	6,260	480	47.10	5,125	6,260					
60	C	2.00	240	47.05	6,195	7,580	480	47.10	6,195	7,580					
60	D	2.38	240	47.05	7,370	9,015	480	47.10	7,370	9,015					

* All weights are rounded out to the nearest 5-lb multiple.

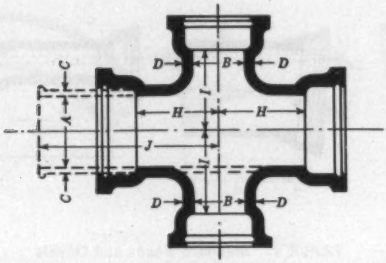


TABLE 5. Standard Tees and Crosses

Nominal Diam. (*A* × *B*) *in.*	Class	Dimensions—*in.*			Weight*—*lb*			
					Tees		Crosses	
		H	*J*	*I*	2 Bells	3 Bells	3 Bells	4 Bells
4×3	D	11	23	11	120	125	150	155
4×4	D	11	23	11	130	130	170	170
6×3	D	12	24	12	175	170	205	200
6×4	D	12	24	12	180	180	220	220
6×6	D	12	24	12	200	200	260	255
8×4	D	13	25	13	250	250	290	290
8×6	D	13	25	13	270	265	325	325
8×8	D	13	25	13	290	285	370	370
10×4	D	14	26	14	340	330	380	370
10×6	D	14	26	14	360	350	415	405
10×8	D	14	26	14	380	370	455	445
10×10	D	14	26	14	405	390	505	495
12×4	D	15	27	15	445	425	485	465
12×6	D	15	27	15	460	440	515	495
12×8	D	15	27	15	485	465	560	540
12×10	D	15	27	15	505	495	610	590
12×12	D	15	27	15	535	515	665	645
14×4	B	16	28	16	495	485	535	525
14×4	D	16	28	16	595	570	635	610
14×6	B	16	28	16	515	505	570	560
14×6	D	16	28	16	610	585	665	640
14×8	B	16	28	16	540	525	620	610
14×8	D	16	28	16	635	610	710	685
14×10	B	16	28	16	560	555	670	660
14×10	D	16	28	16	655	630	760	730

* All weights are rounded out to the nearest 5-lb multiple.

TABLE 5. Standard Tees and Crosses (continued)

Nominal Diam. (A × B) in.	Class	Dimensions—in.			Weight*—lb			
					Tees		Crosses	
		H	J	I	2 Bells	3 Bells	3 Bells	4 Bells
14×12	B	16	28	16	590	580	730	720
14×12	D	16	28	16	680	655	810	785
14×14	B	16	28	16	600	590	745	740
14×14	D	16	28	16	725	695	890	860
16×4	B	17	29	17	620	615	660	655
16×4	D	17	29	17	765	740	800	780
16×6	B	17	29	17	635	630	690	690
16×6	D	17	29	17	775	755	830	805
16×8	B	17	29	17	660	655	740	735
16×8	D	17	29	17	800	775	875	855
16×10	B	17	29	17	685	680	790	785
16×10	D	17	29	17	825	800	925	900
16×12	B	17	29	17	710	710	845	840
16×12	D	17	29	17	850	825	975	950
16×14	B	17	29	17	720	715	860	860
16×14	D	17	29	17	885	860	1,045	1,020
16×16	B	17	29	17	760	755	940	940
16×16	D	17	29	17	935	915	1,150	1,125
18×4	B	18	30	18	755	745	795	785
18×4	D	18	30	18	945	915	985	955
18×6	B	18	30	18	770	765	830	820
18×6	D	18	30	18	955	925	1,010	980
18×8	B	18	30	18	795	785	870	865
18×8	D	18	30	18	980	950	1,055	1,025
18×10	B	18	30	18	820	810	920	915
18×10	D	18	30	18	1,005	975	1,100	1,070
18×12	B	18	30	18	845	835	975	965
18×12	D	18	30	18	1,030	1,000	1,150	1,125
18×14	B	18	30	18	850	845	990	985
18×14	D	18	30	18	1,055	1,025	1,210	1,180
18×16	B	18	30	18	885	880	1,060	1,050
18×16	D	18	30	18	1,105	1,075	1,305	1,275
18×18	B	18	30	18	925	915	1,135	1,130
18×18	D	18	30	18	1,155	1,125	1,405	1,375
20×4	B	19	31	19	910	900	950	940
20×4	D	19	31	19	1,160	1,130	1,195	1,170
20×6	B	19	31	19	930	915	985	975
20×6	D	19	31	19	1,175	1,145	1,225	1,200

* All weights are rounded out to the nearest 5-lb multiple.

TABLE 5. Standard Tees and Crosses (continued)

Nominal Diam. (A × B) *in.*	Class	Dimensions—*in.*			Weight*—*lb*			
					Tees		Crosses	
		H	*J*	*I*	2 Bells	3 Bells	3 Bells	4 Bells
20×8	B	19	31	19	950	940	1,030	1,020
20×8	D	19	31	19	1,195	1,165	1,265	1,240
20×10	B	19	31	19	970	965	1.075	1.065
20×10	D	19	31	19	1,215	1,185	1,310	1,280
20×12	B	19	31	19	1,000	990	1,125	1,115
20×12	D	19	31	19	1,240	1,210	1,355	1,330
20×14	B	19	31	19	1,010	995	1,145	1,130
20×14	D	19	31	19	1,270	1,240	1,420	1,390
20×16	B	19	31	19	1,040	1,030	1,215	1,200
20×16	D	19	31	19	1,315	1,290	1,515	1,485
20×18	B	19	31	19	1,075	1,065	1,280	1,265
20×18	D	19	31	19	1,360	1,330	1,600	1,570
20×20	B	19	31	19	1,115	1,105	1,360	1,350
20×20	D	19	31	19	1,420	1,390	1,720	1,690
24×6	B	21	33	21	1,280	1,260	1,335	1,315
24×6	D	21	33	21	1,655	1,615	1,705	1,665
24×8	B	21	33	21	1,305	1,280	1,380	1,355
24×8	D	21	33	21	1,675	1,630	1,745	1,705
24×10	B	21	33	21	1,325	1,305	1,430	1,410
24×10	D	21	33	21	1,695	1,650	1,785	1,745
24×12	B	21	33	21	1,350	1,330	1,475	1,450
24×12	D	21	33	21	1,715	1,675	1,825	1,785
24×14	B	21	33	21	1,355	1,335	1,485	1,460
24×14	D	21	33	21	1,745	1,700	1,885	1,845
24×16	B	21	33	21	1,390	1,370	1,550	1,530
24×16	D	21	33	21	1,785	1,745	1,975	1,930
24×18	B	21	33	21	1,420	1,395	1,610	1,585
24×18	D	21	33	21	1,825	1,780	2,045	2,005
24×20	B	21	33	21	1,455	1,430	1,685	1,660
24×20	D	21	33	21	1,880	1,835	2,155	2,110
24×24	B	21	33	21	1,530	1,505	1,840	1,815
24×24	D	21	33	21	1,985	1,945	2,370	2,325
30×6	A	13	25	24	1,290	1,320	1,350	1,375
30×6	B	13	25	24	1,450	1,430	1,505	1,485
30×6	C	13	25	24	1,690	1,670	1,745	1,725
30×6	D	13	25	24	1,945	1,935	1,995	1,985
30×8	A	14	26	24	1,365	1,390	1,445	1,470
30×8	B	14	26	24	1,525	1,505	1,600	1,580
30×8	C	14	26	24	1,775	1,755	1,850	1,830
30×8	D	14	26	24	2,040	2,030	2,110	2,100

* All weights are rounded out to the nearest 5-lb multiple.

TABLE 5. Standard Tees and Crosses (continued)

Nominal Diam. ($A \times B$) *in.*	Class	Dimensions—*in.*			Weight*—*lb*			
					Tees		Crosses	
		H	*J*	*I*	2 Bells	3 Bells	3 Bells	4 Bells
30×10	A	15	27	24	1,430	1,460	1,535	1,560
30×10	B	15	27	24	1,605	1,585	1,705	1,685
30×10	C	15	27	24	1,865	1,840	1,955	1,935
30×10	D	15	27	24	2,135	2,125	2,220	2,210
30×12	A	15	27	24	1,460	1,485	1,585	1,615
30×12	B	15	27	24	1,630	1,610	1,750	1,730
30×12	C	15	27	24	1,885	1,865	2,000	1,975
30×12	D	15	27	24	2,155	2,145	2,260	2,250
30×14	A	18	30	26	1,625	1,650	1,775	1,800
30×14	B	18	30	26	1,815	1,795	1,960	1,940
30×14	C	18	30	26	2,135	2,110	2,300	2,280
30×14	D	18	30	26	2,430	2,420	2,590	2,575
30×16	A	19	31	26	1,710	1,735	1,895	1,920
30×16	B	19	31	26	1,910	1,890	2,085	2,065
30×16	C	19	31	26	2,250	2,225	2,460	2,435
30×16	D	19	31	26	2,555	2,540	2,755	2,745
30×18	A	20	34	26	1,840	1,815	2,060	2,035
30×18	B	20	34	26	2,050	1,975	2,260	2,185
30×18	C	20	34	26	2,425	2,335	2,680	2,590
30×18	D	20	34	26	2,745	2,655	2,990	2,900
30×20	A	21	36	26	1,945	1,905	2,205	2,160
30×20	B	21	36	26	2,170	2,070	2,420	2,315
30×20	C	21	36	26	2,580	2,455	2,890	2,770
30×20	D	21	36	26	2,910	2,785	3,205	3,075
30×24	A	23	38	26	2,130	2,080	2,470	2,425
30×24	B	23	38	26	2,365	2,255	2,680	2,575
30×24	C	23	38	26	2,825	2,700	3,250	3,130
30×24	D	23	38	26	3,165	3,040	3,560	3,435
30×30	A	26	43	26	2,445	2,355	2,920	2,825
30×30	B	26	43	26	2,765	2,600	3,260	3,100
30×30	C	26	43	26	3,245	3,055	3,825	3,635
30×30	D	26	43	26	3,745	3,545	4,425	4,225
36×8	A	14	26	27	1,785	1,805	1,865	1,885
36×8	B	14	26	27	2,070	2,090	2,140	2,165
36×8	C	14	26	27	2,420	2,430	2,490	2,500
36×8	D	14	26	27	2,800	2,795	2,865	2,865
36×10	A	15	27	27	1,875	1,895	1,970	1,990
36×10	B	15	27	27	2,165	2,190	2,255	2,280
36×10	C	15	27	27	2,525	2,535	2,615	2,625
36×10	D	15	27	27	2,920	2,920	3,000	3,000

* All weights are rounded out to the nearest 5-lb multiple.

TABLE 5. Standard Tees and Crosses (continued)

Nominal Diam. (*A* × *B*) *in.*	Class	Dimensions—*in.*			Weight*—*lb*			
					Tees		Crosses	
		H	*J*	*I*	2 Bells	3 Bells	3 Bells	4 Bells
36×12	A	16	28	27	1,960	1,985	2,085	2,105
36×12	B	16	28	27	2,260	2,285	2,375	2,400
36×12	C	16	28	27	2,635	2,645	2,745	2,755
36×12	D	16	28	27	3,040	3,040	3,140	3,140
36×14	A	18	30	29	2,110	2,130	2,255	2,275
36×14	B	18	30	29	2,430	2,455	2,565	2,590
36×14	C	18	30	29	2,860	2,865	3,020	3,025
36×14	D	18	30	29	3,295	3,295	3,435	3,435
36×16	A	19	31	29	2,210	2,230	2,390	2,410
36×16	B	19	31	29	2,540	2,560	2,705	2,730
36×16	C	19	31	29	3,000	3,005	3,195	3,205
36×16	D	19	31	29	3,440	3,440	3,625	3,625
36×18	A	20	34	29	2,370	2,325	2,580	2,535
36×18	B	20	34	29	2,715	2,665	2,915	2,860
36×18	C	20	34	29	3,215	3,135	3,455	3,375
36×18	D	20	34	29	3,690	3,585	3,920	3,810
36×20	A	21	36	29	2,505	2,425	2,750	2,670
36×20	B	21	36	29	2,865	2,775	3,095	3,000
36×20	C	21	36	29	3,405	3,275	3,695	3,565
36×20	D	21	36	29	3,890	3,730	4,160	4,000
36×24	A	23	38	29	2,710	2,630	3,025	2,950
36×24	B	23	38	29	3,080	2,990	3,375	3,285
36×24	C	23	38	29	3,685	3,560	4,080	3,950
36×24	D	23	38	29	4,190	4,030	4,545	4,385
36×30	A	26	43	29	3,075	2,930	3,500	3,360
36×30	B	26	43	29	3,535	3,370	3,980	3,815
36×30	C	26	43	29	4,170	3,955	4,685	4,470
36×30	D	26	43	29	4,850	4,585	5,445	5,180
36×36	A	29	46	29	3,430	3,290	4,020	3,875
36×36	B	29	46	29	3,995	3,825	4,675	4,510
36×36	C	29	46	29	4,705	4,490	5,490	5,275
36×36	D	29	46	29	5,465	5,200	6,365	6,100
42×12	A	16	28	30	2,545	2,615	2,660	2,735
42×12	B	16	28	30	2,895	2,935	3,005	3,045
42×12	C	16	28	30	3,470	3,505	3,565	3,605
42×12	D	16	28	30	3,990	4,030	4,080	4,120
42×14	A	18	30	32	2,735	2,805	2,870	2,945
42×14	B	18	30	32	3,110	3,150	3,235	3,275
42×14	C	18	30	32	3,750	3,785	3,895	3,930
42×14	D	18	30	32	4,305	4,345	4,440	4,475

* All weights are rounded out to the nearest 5-lb multiple.

TABLE 5. Standard Tees and Crosses (continued)

Nominal Diam. (A×B) in.	Class	Dimensions—in.			Weight*—lb			
					Tees		Crosses	
		H	J	I	2 Bells	3 Bells	3 Bells	4 Bells
42×16	A	19	31	32	2,850	2,920	3,020	3,090
42×16	B	19	31	32	3,235	3,280	3,395	3,435
42×16	C	19	31	32	3,910	3,945	4,095	4,135
42×16	D	19	31	32	4,485	4,520	4,650	4,690
42×18	A	20	34	32	3,045	3,035	3,245	3,230
42×18	B	20	34	32	3,455	3,400	3,640	3,580
42×18	C	20	34	32	4,180	4,100	4,405	4,325
42×18	D	20	34	32	4,790	4,690	4,990	4,890
42×20	A	21	36	32	3,205	3,150	3,435	3,380
42×20	B	21	36	32	3,635	3,525	3,850	3,740
42×20	C	21	36	32	4,405	4,265	4,675	4,535
42×20	D	21	36	32	5,040	4,870	5,280	5,110
42×24	A	23	38	32	3,430	3,380	3,725	3,675
42×24	B	23	38	32	3,885	3,780	4,160	4,050
42×24	C	23	38	32	4,730	4,595	5,090	4,950
42×24	D	23	38	32	5,390	5,220	5,705	5,535
42×30	A	26	43	32	3,865	3,730	4,260	4,120
41×30	B	26	43	32	4,385	4,205	4,785	4,605
42×30	C	26	43	32	5,310	5,050	5,770	5,510
43×30	D	26	43	32	6,155	5,845	6,685	6,375
42×36	A	29	46	32	4,250	4,115	4,780	4,645
42×36	B	29	46	32	4,910	4,705	5,520	5,315
42×36	C	29	46	32	5,885	5,630	6,575	6,315
42×36	D	29	46	32	6,815	6,510	7,600	7,295
42×42	A	32	49	32	4,745	4,610	5,515	5,380
42×42	B	32	49	32	5,440	5,235	6,290	6,085
42×42	C	32	49	32	6,565	6,310	7,570	7,315
42×42	D	32	49	32	7,595	7,285	8,740	8,430
48×12	A	17	29	33	3,345	3,395	3,455	3,505
48×12	B	17	29	33	3,760	3,815	3,865	3,920
48×12	C	17	29	33	4,520	4,585	4,610	4,675
48×12	D	17	29	33	5,195	5,280	5,275	5,355
48×14	A	18	30	35	3,475	3,520	3,600	3,650
48×14	B	18	30	35	3,900	3,955	4,020	4,075
48×14	C	18	30	35	4,710	4,775	4,845	4,905
48×14	D	18	30	35	5,400	5,485	5,525	5,610
48×16	A	19	31	35	3,615	3,660	3,775	3,820
48×16	B	19	31	35	4,050	4,105	4,200	4,255
48×16	C	19	31	35	4,900	4,960	5,070	5,135
48×16	D	19	31	35	5,615	5,700	5,770	5,855

* All weights are rounded out to the nearest 5-lb multiple.

TABLE 5. Standard Tees and Crosses (continued)

Nominal Diam. ($A \times B$) *in.*	Class	Dimensions—*in.*			Weight*—*lb*			
					Tees		Crosses	
		H	*J*	*I*	2 Bells	3 Bells	3 Bells	4 Bells
48×18	A	20	34	35	3,860	3,795	4,040	3,980
48×18	B	20	34	35	4,320	4,255	4,490	4,420
48×18	C	20	34	35	5,230	5,145	5,435	5,350
48×18	D	20	34	35	5,990	5,900	6,170	6,085
48×20	A	21	36	35	4,055	3,935	4,265	4,145
48×20	B	21	36	35	4,540	4,405	4,735	4,605
48×20	C	21	36	35	5,500	5,335	5,745	5,585
48×20	D	21	36	35	6,285	6,110	6,505	6,330
48×24	A	23	38	35	4,330	4,210	4,595	4,475
48×24	B	23	38	35	4,830	4,700	5,080	4,945
48×24	C	23	38	35	5,875	5,720	6,205	6,045
48×24	D	23	38	35	6,700	6,520	6,985	6,805
48×30	A	26	43	35	4,845	4,620	5,195	4,965
48×30	B	26	43	35	5,445	5,190	5,810	5,555
48×30	C	26	43	35	6,560	6,250	6,970	6,660
48×30	D	26	43	35	7,580	7,230	8,055	7,705
48×36	A	29	46	35	5,280	5,055	5,740	5,515
48×36	B	29	46	35	5,995	5,735	6,535	6,275
48×36	C	29	46	35	7,210	6,900	7,815	7,510
48×36	D	29	46	35	8,320	7,975	9,020	8,675
48×42	A	32	49	35	5,815	5,590	6,480	6,250
48×42	B	32	49	35	6,565	6,305	7,305	7,050
48×42	C	32	49	35	7,925	7,620	8,805	8,490
48×42	D	32	49	35	9,150	8,800	10,150	9,800
48×48	A	35	52	35	6,390	6,165	7,300	7,075
48×48	B	35	52	35	7,220	6,965	8,235	7,980
48×48	C	35	52	35	8,720	8,410	9,945	9,630
48×48	D	35	52	35	10,065	9,715	11,465	11,115

* All weights are rounded out to the nearest 5-lb multiple.

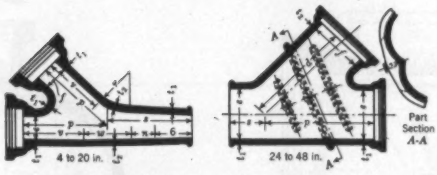
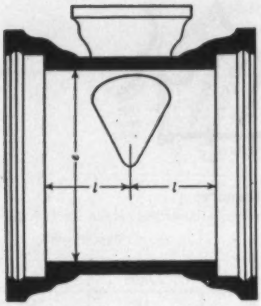


TABLE 6. Standard Y Branches

Size *in.* (*e* × *f*)	Class	Dimensions—*in.*		Weight*—*lb*	
		p	*s*	2 Bells	3 Bells
4 × 4	D	10.50	11.50	110	125
6 × 4	D	13.00	13.00	160	185
6 × 6	D	13.00	13.00	175	200
8 × 4	D	16.00	14.00	235	270
8 × 6	D	16.00	14.00	255	285
8 × 8	D	16.00	14.00	275	310
10 × 6	D	18.50	15.50	355	390
10 × 8	D	18.50	15.50	380	420
10 × 10	D	18.50	15.50	400	445
12 × 6	D	21.50	15.50	470	515
12 × 8	D	21.50	15.50	500	545
12 × 10	D	21.50	15.50	525	570
12 × 12	D	21.50	15.50	555	605
14 × 6	B	24.00	16.00	525	580
14 × 6	D	24.00	16.00	615	670
14 × 8	B	24.00	16.00	555	610
14 × 8	D	24.00	16.00	645	700
14 × 10	B	24.00	16.00	590	640
14 × 10	D	24.00	16.00	675	730
14 × 12	B	24.00	16.00	620	675
14 × 12	D	24.00	16.00	710	765
14 × 14	B	24.00	16.00	625	680
14 × 14	D	24.00	16.00	750	800
16 × 8	B	31.00	17.50	820	890
16 × 8	D	31.00	17.50	995	1,070
16 × 10	B	31.00	17.50	860	935
16 × 10	D	31.00	17.50	1,035	1,110
16 × 12	B	31.00	17.50	905	980
16 × 12	D	31.00	17.50	1,080	1,155
16 × 14	B	31.00	17.50	915	990
16 × 14	D	31.00	17.50	1,130	1,205
16 × 16	B	31.00	17.50	965	1,040
16 × 16	D	31.00	17.50	1,205	1,285
18 × 10	B	34.00	18.00	1,065	1,150
18 × 10	D	34.00	18.00	1,310	1,400
18 × 12	B	34.00	18.00	1,110	1,195
18 × 12	D	34.00	18.00	1,355	1,445
18 × 14	B	34.00	18.00	1,120	1,205
18 × 14	D	34.00	18.00	1,415	1,500
18 × 16	B	34.00	18.00	1,185	1,270
18 × 16	D	34.00	18.00	1,495	1,585
18 × 18	B	34.00	18.00	1,225	1,305
18 × 18	D	34.00	18.00	1,565	1,655
20 × 12	B	37.00	18.75	1,355	1,455
20 × 12	D	37.00	18.75	1,650	1,770
20 × 14	B	37.00	18.75	1,365	1,465
20 × 14	D	37.00	18.75	1,715	1,830
20 × 16	B	37.00	18.75	1,430	1,530
20 × 16	D	37.00	18.75	1,805	1,920
20 × 18	B	37.00	18.75	1,485	1,585
20 × 18	D	37.00	18.75	1,880	1,995
20 × 20	B	37.00	18.75	1,530	1,630
20 × 20	D	37.00	18.75	1,935	2,055

* All weights are rounded out to nearest 5-lb multiple.



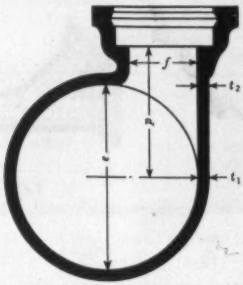
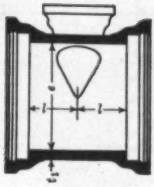


TABLE 7. Standard Blowoff Branches

Size *in.*		Class	Dimensions *in.*				Weight* *lb*	Size *in.*		Class	Dimensions *in.*				Weight* *lb*
e	*f*		*l*	*p*	t_1	t_2		*e*	*f*		*l*	*p*	t_1	t_2	
8	4	D	12	7	0.60	0.52	230	36	12	A	13	23	0.99	0.75	1,800
10	4	D	12	8	0.68	0.52	300	36	12	B	13	23	1.15	0.75	2,065
10	6	D	12	8	0.68	0.55	310	36	12	C	13	23	1.36	0.75	2,375
12	4	D	12	10	0.75	0.52	370	36	12	D	13	23	1.58	0.75	2,720
12	6	D	12	10	0.75	0.55	385								
								42	12	A	15	26	1.10	0.75	2,555
14	4	B	12	11	0.66	0.52	415	42	12	B	15	26	1.28	0.75	2,850
14	4	D	12	11	0.82	0.52	480	42	12	C	15	26	1.54	0.75	3,395
14	6	B	12	11	0.66	0.55	425	42	12	D	15	26	1.78	0.75	3,895
14	6	D	12	11	0.82	0.55	495								
								42	16	A	15	26	1.10	0.70	2,575
16	4	B	12	12	0.70	0.52	510	42	16	B	15	26	1.28	0.70	2,870
16	4	D	12	12	0.89	0.52	605	42	16	C	15	26	1.54	0.89	3,460
16	6	B	12	12	0.70	0.55	525	42	16	D	15	26	1.78	0.89	3,940
16	6	D	12	12	0.89	0.55	620								
								48	12	A	17	30	1.26	0.75	3,435
18	4	B	12	13	0.75	0.52	600	48	12	B	17	30	1.42	0.75	3,840
18	4	D	12	13	0.96	0.52	730	48	12	C	17	30	1.71	0.75	4,610
18	6	B	12	13	0.75	0.55	615	48	12	D	17	30	1.96	0.75	5,300
18	6	D	12	13	0.96	0.55	740								
								48	16	A	17	30	1.26	0.70	3,440
20	4	B	12	14	0.80	0.52	700	48	16	B	17	30	1.42	0.70	3,850
20	4	D	12	14	1.03	0.52	870	48	16	C	17	30	1.71	0.89	4,660
20	6	B	12	14	0.80	0.55	710	48	16	D	17	30	1.96	0.89	5,335
20	6	D	12	14	1.03	0.55	880								
								54	12	A	19	33	1.35	0.75	4,505
24	6	B	12	16	0.89	0.55	920	54	12	B	19	33	1.55	0.75	5,170
24	6	D	12	16	1.16	0.55	1,165	54	12	C	19	33	1.90	0.75	6,245
24	8	B	12	16	0.89	0.60	935	54	12	D	19	33	2.23	0.75	7,295
24	8	D	12	16	1.16	0.60	1,175								
								54	16	A	19	33	1.35	0.70	4,515
30	8	A	13	20	0.88	0.60	1,345	54	16	B	19	33	1.55	0.70	5,175
30	8	B	13	20	1.03	0.60	1,455	54	16	C	19	33	1.90	0.89	6,300
30	8	C	13	20	1.20	0.60	1,690	54	16	D	19	33	2.23	0.89	7,340
30	8	D	13	20	1.37	0.60	1,950								
								60	12	A	21	36	1.39	0.75	5,540
30	12	A	13	20	0.88	0.75	1,385	60	12	B	21	36	1.67	0.75	6,430
30	12	B	13	20	1.03	0.75	1,490	60	12	C	21	36	2.00	0.75	7,740
30	12	C	13	20	1.20	0.75	1,725	60	12	D	21	36	2.38	0.75	9,155
30	12	D	13	20	1.37	0.75	1,980								
								60	16	A	21	36	1.39	0.70	5,555
36	8	A	13	23	0.99	0.60	1,750	60	16	B	21	36	1.67	0.70	6,440
36	8	B	13	23	1.15	0.60	2,025	60	16	C	21	36	2.00	0.89	7,795
36	8	C	13	23	1.36	0.60	2,345	60	16	D	21	36	2.38	0.89	9,185
36	8	D	13	23	1.58	0.60	2,695								

* All weights are rounded out to the nearest 5-lb multiple.



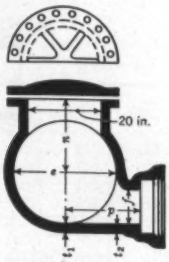


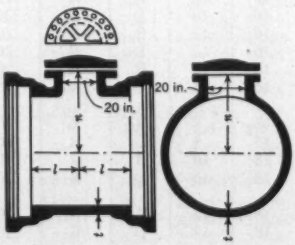
TABLE 8. Standard Blowoff Branches With Manhole

Size in.		Class	Dimensions in.					Weight* lb	Size in.		Class	Dimensions in.					Weight* lb
a	f		l	p	n	t_1	t_2		e	f		l	p	n	t_1	t_2	
30	8	A	21	20	26.0	0.88	0.60	2,240	48	12	A	21	30	35.0	1.26	0.75	4,330
30	8	B	21	20	26.0	1.03	0.60	2,400	48	12	B	21	30	35.0	1.42	0.75	4,790
30	8	C	21	20	26.0	1.20	0.60	2,735	48	12	C	21	30	35.0	1.71	0.75	5,655
30	8	D	21	20	26.0	1.37	0.60	3,060	48	12	D	21	30	35.0	1.96	0.75	6,415
30	12	A	21	20	26.0	0.88	0.75	2,285	48	16	A	21	30	35.0	1.26	0.70	4,345
30	12	B	21	20	26.0	1.03	0.75	2,435	48	16	B	21	30	35.0	1.42	0.70	4,795
30	12	C	21	20	26.0	1.20	0.75	2,770	48	16	C	21	30	35.0	1.71	0.89	5,710
30	12	D	21	20	26.0	1.37	0.75	3,090	48	16	D	21	30	35.0	1.96	0.89	6,455
36	8	A	21	23	29.0	0.99	0.60	2,760	54	12	A	28	33	38.5	1.35	0.75	6,155
36	8	B	21	23	29.0	1.15	0.60	3,110	54	12	B	28	33	38.5	1.55	0.75	6,975
36	8	C	21	23	29.0	1.36	0.60	3,555	54	12	C	28	33	38.5	1.90	0.75	8,370
36	8	D	21	23	29.0	1.58	0.60	4,005	54	12	D	28	33	38.5	2.23	0.75	9,695
36	12	A	21	23	29.0	0.99	0.75	2,805	54	16	A	28	33	38.5	1.35	0.70	6,160
36	12	B	21	23	29.0	1.15	0.75	3,145	54	16	B	28	33	38.5	1.55	0.70	6,985
36	12	C	21	23	29.0	1.36	0.75	3,590	54	16	C	28	33	38.5	1.90	0.89	8,425
36	12	D	21	23	29.0	1.58	0.75	4,035	54	16	D	28	33	38.5	2.23	0.89	9,735
42	12	A	21	26	32.0	1.10	0.75	3,535	60	12	A	28	36	42.0	1.39	0.75	7,065
42	12	B	21	26	32.0	1.28	0.75	3,900	60	12	B	28	36	42.0	1.67	0.75	8,145
42	12	C	21	26	32.0	1.54	0.75	4,570	60	12	C	28	36	42.0	2.00	0.75	9,715
42	12	D	21	26	32.0	1.78	0.75	5,170	60	12	D	28	36	42.0	2.38	0.75	11,400
42	16	A	21	26	32.0	1.10	0.70	3,555	60	16	A	28	36	42.0	1.39	0.70	7,065
42	16	B	21	26	32.0	1.28	0.70	3,910	60	16	B	28	36	42.0	1.67	0.70	8,145
42	16	C	21	26	32.0	1.54	0.89	4,640	60	16	C	28	36	42.0	2.00	0.89	9,770
42	16	D	21	26	32.0	1.78	0.89	5,205	60	16	D	28	36	42.0	2.38	0.89	11 ,445

* All weights are rounded out to the nearest 5-lb multiple. The approximate weight of the manhole cap is 290 lb. Manholes are regularly furnished with blank flange, bolts, nuts, and gasket.

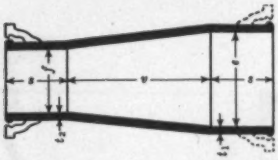
TABLE 9. Standard Manhole Pipe*

Size in.	Class	n in.	l, in.	Weight† lb	Size in.	Class	n in.	l, in.	Weight† lb
30	A	26.0	21	2,155	48	A	35.0	21	4,185
30	B	26.0	21	2,320	48	B	35.0	21	4,655
30	C	26.0	21	2,660	48	C	35.0	21	5,540
30	D	26.0	21	2,990	48	D	35.0	21	6,320
36	A	29.0	21	2,675	54	A	38.5	28	5,995
36	B	29.0	21	3,025	54	B	38.5	28	6,835
36	C	29.0	21	3,485	54	C	38.5	28	8,255
36	D	29.0	21	3,935	54	D	38.5	28	9,600
42	A	32.0	21	3,400	60	A	42.0	28	6,900
42	B	32.0	21	3,775	60	B	42.0	28	8,005
42	C	32.0	21	4,470	60	C	42.0	28	9,595
42	D	32.0	21	5,075	60	D	42.0	28	11,310



* Dimension t is as given in Table 8 for t_1.

† All weights are rounded out to the nearest 5-lb multiple. The weight of manhole pipe, with and without blowoff, includes the 300-lb weight of blank flange, steel nuts and bolts. Manholes are regularly furnished with 20-in. blank flange, bolts, nuts, and gasket.



Reducer (R). Size Range: 6 × 4 to 60 × 54 in.

TABLE 10. Standard Reducers

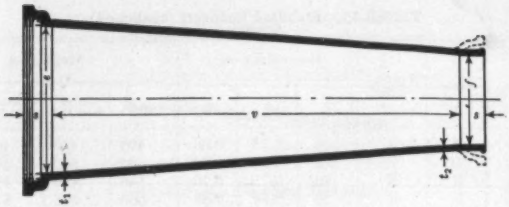
Size *in.*		Class	Reducer Type*	Dimensions†—*in.*			Weight‡—*lb*			
e	*f*			*v*	t_1	t_2	2 Bells	Large End Bell	Small End Bell	Spigot Ends
6	4	D	R	18	0.55	0.52	125	105	100	80
8	4	D	R	18	0.60	0.52	155	135	120	100
8	6	D	R	18	0.60	0.55	180	155	145	120
10	4	D	R	18	0.68	0.52	185	170	145	130
10	6	D	R	18	0.68	0.55	210	190	170	145
10	8	D	R	18	0.68	0.60	245	210	205	170
12	4	D	R	18	0.75	0.52	225	205	175	155
12	6	D	R	18	0.75	0.55	250	225	200	175
12	8	D	R	18	0.75	0.60	280	245	230	200
12	10	D	R	18	0.75	0.68	315	275	270	230
14	6	B	R	20	0.66	0.55	270	250	220	195
14	6	D	R	20	0.82	0.55	305	280	250	225
14	8	B	R	20	0.66	0.60	305	270	250	220
14	8	D	R	20	0.82	0.60	340	305	285	250
14	10	B	R	20	0.66	0.68	345	305	290	250
14	10	D	R	20	0.82	0.68	380	340	325	285
14	12	B	R	20	0.66	0.75	385	335	330	280
14	12	D	R	20	0.82	0.75	420	370	365	320
16	6	B	R	20	0.70	0.55	320	295	245	220
16	6	D	R	20	0.89	0.55	365	340	290	265
16	8	B	R	20	0.70	0.60	355	320	280	250
16	8	D	R	20	0.89	0.60	400	370	325	290
16	10	B	R	20	0.70	0.68	395	350	320	280
16	10	D	R	20	0.89	0.68	440	400	365	325
16	12	B	R	20	0.70	0.75	435	385	360	310
16	12	D	R	20	0.89	0.75	485	435	405	360
16	14	B	R	20	0.70	0.66	445	390	370	320
16	14	D	R	20	0.89	0.82	535	480	460	405
18	8	B	R	20	0.75	0.60	400	365	315	280
18	8	D	R	20	0.96	0.60	460	430	370	335
18	10	B	R	20	0.75	0.68	440	395	355	310
18	10	D	R	20	0.96	0.68	500	460	410	370
18	12	B	R	20	0.75	0.75	480	430	395	345
18	12	D	R	20	0.96	0.75	545	495	450	405

* Reducer type illustrated on p. 18 indicated by "R"; long reducer shown on p. 20 by "L"; short reducer (p. 23) by "S."
† For all sizes $S = 8$ in.
‡ All weights are rounded out to the nearest 5-lb. multiple.

TABLE 10. Standard Reducers (continued)

Size *in.*		Class	Reducer Type*	Dimensions†—*in.*			Weight‡—*lb*			
e	*f*			*p*	t_1	t_2	2 Bells	Large End Bell	Small End Bell	Spigot Ends
18	14	B	R	20	0.75	0.66	490	435	405	350
18	14	D	R	20	0.96	0.82	595	540	505	450
18	16	B	R	20	0.75	0.70	540	470	455	385
18	16	D	R	20	0.96	0.89	660	585	570	495
20	10	B	R	26	0.80	0.68	545	505	445	405
20	10	D	R	26	1.03	0.68	640	600	525	485
20	12	B	R	26	0.80	0.75	595	545	495	450
20	12	D	R	26	1.03	0.75	690	645	575	525
20	14	B	R	26	0.80	0.66	605	550	505	455
20	14	D	R	26	1.03	0.82	750	695	635	580
20	16	B	R	26	0.80	0.70	660	590	565	490
20	16	D	R	26	1.03	0.89	825	750	710	630
20	18	B	R	26	0.80	0.75	715	630	615	530
20	18	D	R	26	1.03	0.96	895	805	780	690
24	14	B	R	26	0.89	0.66	720	665	595	545
24	14	D	R	26	1.16	0.82	910	855	755	700
24	16	B	R	26	0.89	0.70	780	705	655	580
24	16	D	R	26	1.16	0.89	985	910	835	755
24	18	B	R	26	0.89	0.75	830	745	710	625
24	18	D	R	26	1.16	0.96	1,060	965	905	815
24	20	B	R	26	0.89	0.80	895	795	770	670
24	20	D	R	26	1.16	1.03	1,145	1,030	990	875
30	18	A	R	26	0.88	0.75	985	900	790	710
30	18	B	R	26	1.03	0.75	1,045	960	865	780
30	18	C	R	26	1.20	0.96	1,250	1,160	1,045	950
30	18	D	R	26	1.37	0.96	1,385	1,290	1,130	1,040
30	20	A	S	26	0.88	0.80	1,045	945	855	755
30	20	B	S	26	1.03	0.80	1,105	1,010	930	830
30	20	C	S	26	1.20	1.03	1,340	1,225	1,130	1,015
30	20	D	S	26	1.37	1.03	1,475	1,355	1,220	1,105
30	20	A	L	66	0.88	0.80	1,755	1,660	1,565	1,465
30	20	B	L	66	1.03	0.80	1,885	1,790	1,710	1,610
30	20	C	L	66	1.20	1.03	2,295	2,180	2,085	1,970
30	20	D	L	66	1.37	1.03	2,510	2,395	2,255	2,140
30	24	A	S	26	0.88	0.89	1,170	1,045	975	855
30	24	B	S	26	1.03	0.89	1,230	1,110	1,055	930
30	24	C	S	26	1.20	1.16	1,510	1,360	1,300	1,150
30	24	D	S	26	1.37	1.16	1,645	1,495	1,390	1,240
30	24	A	L	66	0.88	0.89	1,975	1,850	1,785	1,660
30	24	B	L	66	1.03	0.89	2,110	1,985	1,935	1,810
30	24	C	L	66	1.20	1.16	2,595	2,445	2,390	2,235
30	24	D	L	66	1.37	1.16	2,820	2,665	2,565	2,410

* Reducer type illustrated on p. 18 indicated by "R"; long reducer shown on p. 20 by "L"; short reducer (p. 23) by "S."
† For all sizes $S = 8$ in.
‡ All weights are rounded out to the nearest 5-lb multiple.



Long Reducer (L). Size Range: 30 × 20 to 60 × 54 in.

TABLE 10. Standard Reducers (continued)

Size *in.*		Class	Reducer Type*	Dimensions†—*in.*			Weight‡—*lb*			
e	*f*			*v*	t_1	t_2	2 Bells	Large End Bell	Small End Bell	Spigot Ends
36	20	A	S	32	0.99	0.80	1,380	1,280	1,135	1,040
36	20	B	S	32	1.15	0.80	1,530	1,430	1,245	1,145
36	20	C	S	32	1.36	1.03	1,840	1,725	1,520	1,405
36	20	D	S	32	1.58	1.03	2,035	1,920	1,670	1,555
36	20	A	L	66	0.99	0.80	2,105	2,005	1,860	1,765
36	20	B	L	66	1.15	0.80	2,325	2,230	2,045	1,945
36	20	C	L	66	1.36	1.03	2,820	2,705	2,500	2,385
36	20	D	L	66	1.58	1.03	3,120	3,005	2,755	2,640
36	24	A	S	32	0.99	0.89	1,520	1,400	1,280	1,155
36	24	B	S	32	1.15	0.89	1,675	1,550	1,390	1,265
36	24	C	S	32	1.36	1.16	2,040	1,885	1,715	1,565
36	24	D	S	32	1.58	1.16	2,235	2,085	1,875	1,720
36	24	A	L	66	0.99	0.89	2,335	2,210	2,090	1,965
36	24	B	L	66	1.15	0.89	2,560	2,435	2,275	2,155
36	24	C	L	66	1.36	1.16	3,135	2,985	2,815	2,665
36	24	D	L	66	1.58	1.16	3,440	3,290	3,080	2,930
36	30	A	S	32	0.99	0.88	1,690	1,500	1,450	1,255
36	30	B	S	32	1.15	1.03	1,930	1,755	1,645	1,470
36	30	C	S	32	1.36	1.20	2,265	2,055	1,945	1,735
36	30	D	S	32	1.58	1.37	2,630	2,375	2,265	2,010
36	30	A	L	66	0.99	0.89	2,575	2,380	2,330	2,140
36	30	B	L	66	1.15	1.03	2,960	2,785	2,680	2,500
36	30	C	L	66	1.36	1.20	3,485	3,275	3,165	2,955
36	30	D	L	66	1.58	1.37	4,040	3,790	3,680	3,425
42	20	A	S	32	1.10	0.80	1,675	1,575	1,335	1,235
42	20	B	S	32	1.28	0.80	1,825	1,725	1,470	1,375
42	20	C	S	32	1.54	1.03	2,235	2,120	1,815	1,700
42	20	D	S	32	1.78	1.03	2,485	2,370	2,005	1,890
42	20	A	L	66	1.10	0.80	2,530	2,430	2,190	2,095
42	20	B	L	66	1.28	0.80	2,775	2,675	2,420	2,325
42	20	C	L	66	1.54	1.03	3,415	3,300	2,990	2,875
42	20	D	L	66	1.78	1.03	3,785	3,670	3,305	3,190

* Reducer type illustrated on p. 18 indicated by "R"; long reducer shown on p. 20 by "L"; short reducer (p. 23) by "S."
† For all sizes, S = 8 in.
‡ All weights are rounded out to the nearest 5-lb multiple.

TABLE 10. Standard Reducers (continued)

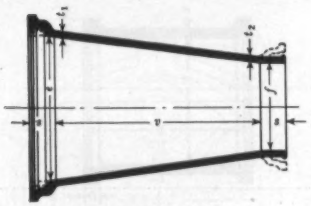
Size *in.*		Class	Reducer Type*	Dimensions†—*in.*			Weight‡—*lb*			
e	*f*			*r*	t_1	t_2	2 Bells	Large End Bell	Small End Bell	Spigot Ends
42	24	A	S	32	1.10	0.89	1,820	1,695	1,480	1,355
42	24	B	S	32	1.28	0.89	1,975	1,850	1,620	1,495
42	24	C	S	32	1.54	1.16	2,440	2,290	2,015	1,865
42	24	D	S	32	1.78	1.16	2,690	2,540	2,210	2,060
42	24	A	L	66	1.10	0.89	2,770	2,645	2,430	2,305
42	24	B	L	66	1.28	0.89	3,020	2,895	2,665	2,540
42	24	C	L	66	1.54	1.16	3,745	3,590	3,320	3,170
42	24	D	L	66	1.78	1.16	4,125	3,975	3,645	3,495
42	30	A	S	32	1.10	0.88	1,990	1,800	1,650	1,460
42	30	B	S	32	1.28	1.03	2,235	2,060	1,885	1,705
42	30	C	S	32	1.54	1.20	2,670	2,465	2,250	2,040
42	30	D	S	32	1.78	1.37	3,095	2,840	2,615	2,360
42	30	A	L	66	1.10	0.88	3,015	2,820	2,675	2,480
42	30	B	L	66	1.28	1.03	3,435	3,255	3,080	2,905
42	30	C	L	66	1.54	1.20	4,105	3,895	3,680	3,475
42	30	D	L	66	1.78	1.37	4,750	4,495	4,270	4,015
42	36	A	S	32	1.10	0.99	2,240	1,995	1,900	1,655
42	36	B	S	32	1.28	1.15	2,570	2,285	2,215	1,930
42	36	C	S	32	1.54	1.36	3,060	2,740	2,635	2,315
42	36	D	S	32	1.78	1.58	3,540	3,180	3,060	2,700
42	36	A	L	66	1.10	0.99	3,400	3,155	3,060	2,820
42	36	B	L	66	1.28	1.15	3,925	3,640	3,570	3,285
42	36	C	L	66	1.54	1.36	4,690	4,370	4,270	3,945
42	36	D	L	66	1.78	1.58	5,445	5,080	4,960	4,600
48	30	A	S	66	1.26	0.88	3,525	3,335	3,125	2,935
48	30	B	S	66	1.42	1.03	3,990	3,815	3,540	3,360
48	30	C	S	66	1.71	1.20	4,780	4,570	4,235	4,025
48	30	D	S	66	1.96	1.37	5,530	5,275	4,890	4,635
48	30	A	L	132	1.26	0.88	5,870	5,675	5,470	5,275
48	30	B	L	132	1.42	1.03	6,675	6,500	6,225	6,050
48	30	C	L	132	1.71	1.20	7,995	7,785	7,450	7,240
48	30	D	L	132	1.96	1.37	9,230	8,975	8,590	8,335
48	36	A	S	66	1.26	0.99	3,930	3,685	3,530	3,285
48	36	B	S	66	1.42	1.15	4,495	4,215	4,045	3,760
48	36	C	S	66	1.71	1.36	5,385	5,065	4,840	4,520
48	36	D	S	66	1.96	1.58	6,245	5,880	5,605	5,240
48	36	A	L	132	1.26	0.99	6,560	6,315	6,155	5,915
48	36	B	L	132	1.42	1.15	7,510	7,225	7,055	6,770
48	36	C	L	132	1.71	1.36	9,005	8,685	8,460	8,140
48	36	D	L	132	1.96	1.58	10,445	10,080	9,805	9,440

* Reducer type illustrated on p. 18 indicated by "R"; long reducer shown on p. 20 by "L"; short reducer (p. 23) by "S."
† For all sizes, $S = 8$ in.
‡ All weights are rounded out to the nearest 5-lb multiple.

TABLE 10. Standard Reducers (continued)

Size *in.*		Class	Reduced Type*	Dimensions†—*in.*			Weight‡—*lb*			
e	*f*			*g*	t_1	t_2	2 Bells	Large End Bell	Small End Bell	Spigot Ends
48	42	A	S	66	1.26	1.10	4,410	4,070	4,010	3,670
48	42	B	S	66	1.42	1.28	5,010	4,655	4,560	4,205
48	42	C	S	66	1.71	1.54	6,065	5,640	5,515	5,095
48	42	D	S	66	1.96	1.78	7,015	6,535	6,375	5,895
48	42	A	L	132	1.26	1.10	7,350	7,010	6,950	6,610
48	42	B	L	132	1.42	1.28	8,385	8,030	7,935	7,580
48	42	C	L	132	1.71	1.54	10,150	9,730	9,605	9,185
48	42	D	L	132	1.96	1.78	11,750	11,270	11,110	10,630
54	36	A	S	66	1.35	0.99	4,435	4,190	3,925	3,680
54	36	B	S	66	1.55	1.15	5,145	4,860	4,545	4,260
54	36	C	S	66	1.90	1.36	6,205	5,885	5,500	5,180
54	36	D	S	66	2.23	1.58	7,270	6,910	6,450	6,085
54	36	A	L	132	1.35	0.99	7,375	7,130	6,865	6,620
54	36	B	L	132	1.55	1.15	8,550	8,265	7,950	7,665
54	36	C	L	132	1.90	1.36	10,345	10,025	9,640	9,320
54	36	D	L	132	2.23	1.58	12,140	11,775	11,320	10,955
54	42	A	S	66	1.35	1.10	4,925	4,585	4,415	4,075
54	42	B	S	66	1.55	1.28	5,675	5,315	5,075	4,720
54	42	C	S	66	1.90	1.54	6,900	6,480	6,200	5,775
54	42	D	S	66	2.23	1.78	8,070	7,590	7,250	6,770
54	42	A	L	132	1.35	1.10	8,190	7,850	7,675	7,335
54	42	B	L	132	1.55	1.28	9,455	9,100	8,855	8,500
54	42	C	L	132	1.90	1.54	11,535	11,110	10,830	10,405
54	42	D	L	132	2.23	1.78	13,500	13,015	12,675	12,195
54	48	A	S	66	1.35	1.26	5,500	5,100	4,985	4,585
54	48	B	S	66	1.55	1.42	6,290	5,835	5,690	5,240
54	48	C	S	66	1.90	1.71	7,655	7,110	6,950	6,405
54	48	D	S	66	2.23	1.96	8,935	8,300	8,115	7,475
54	48	A	L	132	1.35	1.26	9,180	8,780	8,665	8,265
54	48	B	L	132	1.55	1.42	10,495	10,040	9,895	9,440
54	48	C	L	132	1.90	1.71	12,800	12,255	12,095	11,550
54	48	D	L	132	2.23	1.96	14,940	14,300	14,120	13,480
60	36	A	S	66	1.39	0.99	4,885	4,640	4,255	4,010
60	36	B	S	66	1.67	1.15	5,740	5,455	5,065	4,780
60	36	C	S	66	2.00	1.36	6,885	6,565	6,060	5,735
60	36	D	S	66	2.38	1.58	8,135	7,770	7,160	6,800
60	36	A	L	132	1.39	0.99	8,085	7,840	7,455	7,210
60	36	B	L	132	1.67	1.15	9,555	9,270	8,880	8,595
60	36	C	L	132	2.00	1.36	11,460	11,140	10,635	10,315
60	36	D	L	132	2.38	1.58	13,560	13,200	12,590	12,225
60	42	A	S	66	1.39	1.10	5,380	5,040	4,750	4,410
60	42	B	S	66	1.67	1.28	6,280	5,925	5,610	5,255

* Reducer type illustrated on p. 18 indicated by "R"; long reducer shown on p. 20 by "L"; short reducer (p. 23) by "S."
† For all sizes, S = 8 in.
‡ All weights are rounded out to the nearest 5-lb multiple.



Short Reducer (S). Size Range: 30 × 20 to 60 × 54 in.

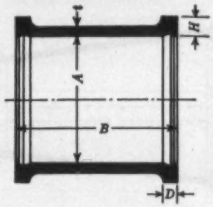
TABLE 10. Standard Reducers (continued)

Size *in.*		Class	Reducer Type*	Dimensions†—*in.*			Weight‡—*lb*			
e	*f*			*v*	t_1	t_2	2 Bells	Large End Bell	Small End Bell	Spigot Ends
60	42	C	S	66	2.00	1.54	7,600	7,175	6,770	6,350
60	42	D	S	66	2.38	1.78	8,955	8,475	7,980	7,500
60	42	A	L	132	1.39	1.10	8,915	8,575	8,285	7,945
60	42	B	L	132	1.67	1.28	10,485	10,135	9,815	9,460
60	42	C	L	132	2.00	1.54	12,680	12,260	11,855	11,435
60	42	D	L	132	2.38	1.78	14,960	14,480	13,990	13,510
60	48	A	S	66	1.39	1.26	5,965	5,565	5,335	4,935
60	48	B	S	66	1.67	1.42	6,915	6,460	6,240	5,785
60	48	C	S	66	2.00	1.71	8,365	7,820	7,540	6,995
60	48	D	S	66	2.38	1.96	9,840	9,200	8,865	8,230
60	48	A	L	132	1.39	1.26	9,925	9,525	9,295	8,895
60	48	B	L	132	1.67	1.42	11,555	11,100	10,880	10,430
60	48	C	L	132	2.00	1.71	13,975	13,430	13,150	12,605
60	48	D	L	132	2.38	1.96	16,440	15,800	15,470	14,830
60	54	A	S	66	1.39	1.35	6,515	6,005	5,885	5,370
60	54	B	S	66	1.67	1.55	7,615	7,020	6,945	6,345
60	54	C	S	66	2.00	1.90	9,265	8,560	8,435	7,730
60	54	D	S	66	2.38	2.23	10,970	10,150	10,000	9,180
60	54	A	L	132	1.39	1.35	10,825	10,315	10,195	9,685
60	54	B	L	132	1.67	1.55	12,710	12,110	12,035	11,440
60	54	C	L	132	2.00	1.90	15,475	14,770	14,645	13,940
60	54	D	L	132	2.38	2.23	18,345	17,525	17,375	16,550

* Reducer type illustrated on p. 18 indicated by "R"; long reducer shown on p. 20 by "L"; short reducer (p. 23) by "S."

† For all sizes, S = 8 in.

‡ All weights are rounded out to the nearest 5-lb multiple.

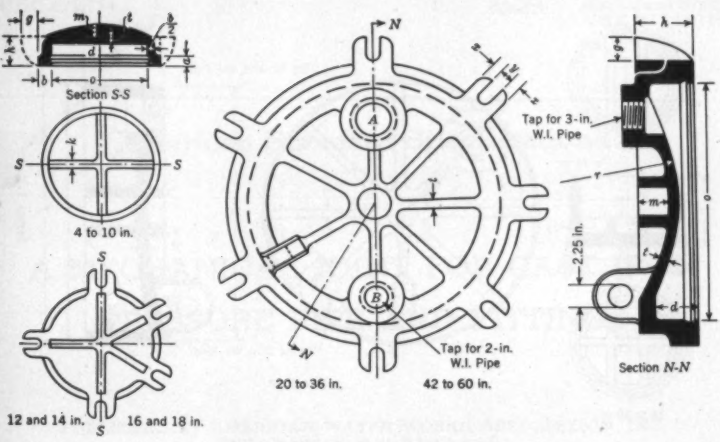


For Dimensions *D* and *H*, see Table 1.

TABLE 11. Standard Sleeves

Size *in.*	Class	Dimensions *in.*			Weight* *lb*	Size *in.*	Class	Dimensions *in.*			Weight* *lb*
		A	*B*	*t*				*A*	*B*	*t*	
4	D	5.80	10	0.65	45	36	B	39.40	15	1.40	920
4	D	5.80	15	0.65	65	36	C	39.80	15	1.60	1,055
6	D	7.90	10	0.70	65	36	D	40.20	15	1.80	1,195
6	D	7.90	15	0.70	90						
						36	A	39.00	24	1.25	1,175
8	D	10.10	12	0.75	100	36	B	39.40	24	1.40	1,340
8	D	10.10	15	0.75	120	36	C	39.80	24	1.60	1,540
10	D	12.20	12	0.80	130	36	D	40.20	24	1.80	1,750
10	D	12.20	18	0.80	180						
12	D	14.30	14	0.85	185	42	A	45.30	15	1.40	1,050
12	D	14.30	18	0.85	225	42	B	45.60	15	1.50	1,140
						42	C	46.20	15	1.75	1,340
14	B	16.20	15	0.85	215	42	D	46.70	15	1.95	1,530
14	B	16.20	18	0.85	255						
14	D	16.50	15	0.90	235	42	A	45.30	24	1.40	1,530
14	D	16.50	18	0.90	275	42	B	45.60	24	1.50	1,660
						42	C	46.20	24	1.75	1,960
16	B	18.50	15	0.90	270	42	D	46.70	24	1.95	2,230
16	B	18.50	24	0.90	400						
16	D	18.90	15	1.00	300	48	A	51.60	15	1.50	1,280
16	D	18.90	24	1.00	445	48	B	51.90	15	1.65	1,435
						48	C	52.50	15	1.95	1,710
18	B	20.60	15	0.95	320	48	D	53.10	15	2.20	1,950
18	B	20.60	24	0.95	470						
18	D	21.00	15	1.05	360	48	A	51.60	24	1.50	1,865
18	D	21.00	24	1.05	530	48	B	51.90	24	1.65	2,080
						48	C	52.50	24	1.95	2,490
20	B	22.70	15	1.00	370	48	D	53.10	24	2.20	2,845
20	B	22.70	24	1.00	540						
20	D	23.10	15	1.15	435	54	A	57.70	15	1.60	1,580
20	D	23.10	24	1.15	640	54	B	58.20	15	1.80	1,800
						54	C	58.90	15	2.15	2,115
24	B	26.90	15	1.05	470	54	D	59.50	15	2.45	2,410
24	B	26.90	24	1.05	685						
24	D	27.40	15	1.25	575	54	A	57.70	24	1.60	2,280
24	D	27.40	24	1.25	840	54	B	58.20	24	1.80	2,595
						54	C	58.90	24	2.15	3,080
30	A	32.80	15	1.15	625	54	D	59.50	24	2.45	3,525
30	B	33.10	15	1.15	630						
30	C	33.50	15	1.32	735	60	A	63.90	15	1.70	1,850
30	D	33.80	15	1.50	860	60	B	64.50	15	1.90	2,075
						60	C	65.30	15	2.25	2,455
30	A	32.80	24	1.15	910	60	D	65.90	15	2.60	2,850
30	B	33.10	24	1.15	920						
30	C	33.50	24	1.32	1,075	60	A	63.90	24	1.70	2,670
30	D	33.80	24	1.50	1,250	60	B	64.50	24	1.90	3,000
						60	C	65.30	24	2.25	3,570
36	A	39.00	15	1.25	810	60	D	65.90	24	2.60	4,150

* All weights are rounded out to the nearest 5-lb multiple.

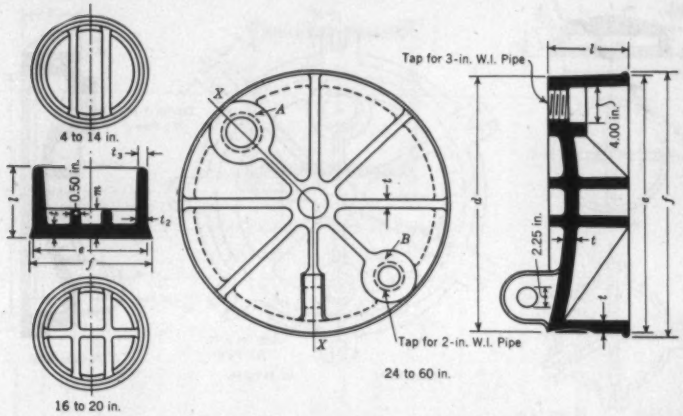


Bosses A and B cast on only when so ordered

TABLE 12. Standard Caps

Size, in.	Class	Dimensions*—in.							Approx. Weight—lb	
		d	o	l	t	m	k	r	Without Lugs	With Lugs
4	D	4.00	5.70	....	0.60	....	....	.....	25	...
6	D	4.00	7.80	....	0.65	....	....	.....	40	...
8	D	4.00	10.00	....	0.75	....	....	.....	60	...
10	D	4.00	12.10	....	0.75	1.50	0.75	16.20	85	...
12	D	4.00	14.20	....	0.75	1.75	0.75	18.70	110	140
14	B	4.00	16.10	....	0.90	1.90	0.75	22.40	135	165
14	D	4.00	16.45	....	0.90	1.90	0.75	22.40	150	180
16	B	4.00	18.40	....	1.00	2.00	0.75	27.00	185	230
16	D	4.00	18.80	....	1.00	2.00	0.75	27.00	200	245
18	B	4.00	20.50	....	1.00	2.00	1.00	32.00	230	280
18	D	4.00	20.92	....	1.00	2.00	1.00	32.90	250	300
20	B	4.00	22.60	....	1.00	3.00	1.00	18.20	280	330
20	D	4.00	23.06	....	1.00	3.00	1.00	18.20	310	360
24	B	4.00	26.80	2.50	1.05	3.50	1.00	23.50	390	440
24	D	4.00	27.32	2.50	1.05	3.50	1.00	23.50	440	490
30	A	4.50	32.74	2.62	1.15	3.50	1.15	34.80	590	665
30	B	4.50	33.00	2.62	1.15	3.50	1.15	34.80	595	670
30	C	4.50	33.40	2.62	1.15	3.50	1.15	34.80	645	725
30	D	4.50	33.74	2.62	1.15	3.50	1.15	34.80	700	785
36	A	4.50	38.96	3.12	1.25	4.00	1.25	44.00	845	925
36	B	4.50	39.30	3.12	1.30	3.95	1.25	44.00	915	995
36	C	4.50	39.70	3.12	1.35	3.90	1.25	44.00	1,000	1,085
36	D	4.50	40.16	3.12	1.40	3.85	1.25	44.00	1,085	1,170
42	A	5.00	45.20	3.37	1.40	4.00	1.40	63.50	1,275	1,395
42	B	5.00	45.50	3.37	1.50	3.90	1.40	63.50	1,395	1,520
42	C	5.00	46.10	3.37	1.60	3.80	1.40	63.50	1,545	1,675
42	D	5.00	46.58	3.37	1.70	3.70	1.40	63.50	1,685	1,820
48	A	5.00	51.50	3.62	1.70	4.00	1.50	76.50	1,790	1,915
48	B	5.00	51.80	3.62	1.90	3.80	1.50	76.50	1,945	2,075
48	C	5.00	52.40	3.62	2.00	3.70	1.50	76.50	2,140	2,275
48	D	5.00	52.98	3.62	2.10	3.60	1.50	76.50	2,335	2,475
54	A	5.50	57.66	3.87	1.90	4.50	1.50	82.00	2,375	2,515
54	B	5.50	58.10	3.87	2.00	4.40	1.50	82.00	2,555	2,700
54	C	5.50	58.80	3.87	2.10	4.30	1.50	82.00	2,800	2,950
54	D	5.50	59.40	3.87	2.20	4.20	1.50	82.00	3,045	3,195
60	A	5.50	63.80	4.12	2.00	4.50	1.50	99.00	2,900	3,045
60	B	5.50	64.40	4.12	2.10	4.40	1.50	99.00	3,105	3,250
60	C	5.50	65.20	4.12	2.20	4.30	1.50	99.00	3,395	3,545
60	D	5.50	65.82	4.12	2.30	4.20	1.50	99.00	3,680	3,835

* Dimension h equals d + t.



Bosses A and B cast on only when so ordered

TABLE 13. Standard Plugs

Size *in.*	Class	Dimensions—*in.*								Number of Ribs	Approx. Weight *lb*
		e	*f*	*d*	*l*	*m*	*t*	t_2	t_3		
4	D	4.90	5.28	.....	5.50	....	0.50	0.40	0.20	..	8
6	D	7.00	7.38	.....	5.50	....	0.60	0.40	0.20	..	14
8	D	9.15	9.65	.....	5.50	2.00	0.60	0.40	0.20	2	25
10	D	11.20	11.70	.....	6.00	2.00	0.70	0.50	0.20	2	40
12	D	13.30	13.80	.....	6.00	2.00	0.75	0.50	0.20	2	50
14	B	15.30	15.80	.....	6.00	2.00	0.70	0.50	0.20	2	60
14	D	15.55	16.15	.....	6.00	2.00	0.75	0.50	0.20	2	65
16	B	17.40	17.90	.....	6.50	2.00	0.70	0.50	0.30	3	90
16	D	17.80	18.30	.....	6.50	2.00	0.80	0.60	0.30	3	95
18	B	19.50	20.00	.....	6.50	2.50	0.75	0.60	0.30	3	110
18	D	19.92	20.42	.....	6.50	2.50	0.85	0.60	0.30	3	120
20	B	21.60	22.10	.....	6.50	2.75	0.85	0.60	0.30	3	150
20	D	22.06	22.56	.....	6.50	2.75	1.00	0.60	0.30	3	155
24	B	25.92	26.30	25.68	8.00	....	0.89	....	....	4	375
24	D	26.44	26.82	26.20	8.00	....	1.16	....	....	4	470
30	A	31.86	32.24	31.62	8.00	....	0.88	....	....	4	480
30	B	32.12	32.50	31.88	8.00	....	1.03	....	....	4	555
30	C	32.52	32.90	32.28	8.00	....	1.20	....	....	4	640
30	D	32.86	33.24	32.62	8.00	....	1.37	....	....	4	725
36	A	38.08	38.46	37.84	8.00	....	0.99	....	....	4	680
36	B	38.42	38.80	38.18	8.00	....	1.15	....	....	4	785
36	C	38.82	39.20	38.58	8.00	....	1.36	....	....	4	915
36	D	39.28	39.66	39.04	8.00	....	1.58	....	....	4	1,050
42	A	44.32	44.70	44.08	9.00	....	1.10	....	....	4	990
42	B	44.62	45.00	44.38	9.00	....	1.28	....	....	4	1,140
42	C	45.22	45.60	44.98	9.00	....	1.54	....	....	4	1,355
42	D	45.70	46.08	45.46	9.00	....	1.78	....	....	4	1,550
48	A	50.62	51.00	50.38	9.00	....	1.26	....	....	4	1,340
48	B	50.92	51.30	50.68	9.00	....	1.42	....	....	4	1,505
48	C	51.52	51.90	51.28	9.00	....	1.71	....	....	4	1,800
48	D	52.10	52.48	51.86	9.00	....	1.96	....	....	4	2,045
54	A	56.78	57.16	56.54	9.00	....	1.35	....	....	4	1,695
54	B	57.22	57.60	56.98	9.00	....	1.55	....	....	4	1,945
54	C	57.92	58.30	57.68	9.00	....	1.90	....	....	4	2,355
54	D	58.52	58.90	58.28	9.00	....	2.23	....	....	4	2,735
60	A	62.92	63.30	62.68	9.00	....	1.39	....	....	4	2,045
60	B	63.52	63.90	63.28	9.00	....	1.67	....	....	4	2,435
60	C	64.32	64.70	64.08	9.00	....	2.00	....	....	4	2,905
60	D	64.94	65.32	64.70	9.00	....	2.38	....	....	4	3,395

UDC 621.774.1

ASA A21.11-1953
(AWWA C111-53)

American Standard Specifications

for

A MECHANICAL JOINT FOR CAST IRON PRESSURE PIPE AND FITTINGS

PUBLISHED BY AMERICAN WATER WORKS ASSOCIATION, INC.

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UDC 621.774.1

ASA A21.11-1953
(AWWA C111-53)

American Standard Specifications for

A Mechanical Joint for Cast Iron Pressure Pipe and Fittings

This specification covers a mechanical joint, in sizes 3 in. to 48 in., for cast iron pressure pipe made by pit cast or by centrifugal methods and cast iron pressure fittings, for gas, water, and other liquids.

Sec. 11–1—Definitions

Purchaser. Wherever the word "purchaser" is used herein, it shall be understood to mean the actual purchaser of the pipe or his authorized agents acting within the scope of the duties entrusted to them.

Manufacturer. The word "manufacturer" means the producer who is making the pipe or fittings and who assumes the responsibility for the fulfillment of these specifications. Accessories, including glands and bolts, may be sublet to other manufacturers. Gaskets will in general be made by other manufacturers who are not expected to permit inspection of manufacturing processes without special arrangements.

Sec. 11–2—Description of Joint

The mechanical joint is a bolted joint of the stuffing-box type, as shown in Fig. 11.1. Each joint shall consist of: [1] a bell cast integrally with the pipe or fitting and provided with an exterior flange having cored or drilled bolt holes and interior annular recesses for the sealing gasket and the spigot of the pipe or fitting; [2] a pipe or fitting spigot; [3] a sealing gasket; [4] a separate cast iron follower gland having cored or drilled bolt holes; and [5] cast iron tee-head bolts and hexagon nuts.

The joint is designed to permit normal expansion, contraction, and deflection of the pipeline.

Sec. 11–3—Pipe and Fittings to Which Mechanical Joint Is Applicable

The mechanical joint made under these specifications is intended for use on cast iron pressure pipe and fittings made in accordance with the following ASA standard specifications and shall be suitable for leakproof assembly therewith:

ASA A21.2 (AWWA C102)—Specifications for Cast Iron Pit Cast Pipe for Water or Other Liquids

ASA A21.3—Specifications for Cast Iron Pit Cast Pipe for Gas

ASA A21.6 (AWWA C106)—Specifications for Cast Iron Pipe Centrifugally Cast in Metal Molds, for Water or Other Liquids

ASA A21.7—Specifications for Cast Iron Pipe Centrifugally Cast in Metal Molds, for Gas

ASA A21.8 (AWWA C108)—Specifications for Cast Iron Pipe Centrifugally Cast in Sand-lined Molds, for Water or Other Liquids

ASA A21.9—Specifications for Cast Iron Pipe Centrifugally Cast in Sand-lined Molds, for Gas

ASA A21.10 (AWWA C110)—Specifications for Short-Body Cast Iron Fittings, 3 Inch to 12 Inch, for 250-psi Water Pressure Plus Water Hammer.

The mechanical joint shall meet all applicable requirements of the above listed specifications and shall have the same pressure rating as the pipe or fitting of which it is a part.

Mechanical joints for pipe or fittings beyond the scope of such standard specifications shall be subject to agreement between the manufacturer and the purchaser.

Sec. 11–4—Dimensions and Tolerances

The joint and accessories shall meet the dimensions and the tolerances shown above them in Table 11.1.

Sec. 11–5—Bell and Socket

The bell shall be cast integrally with the pipe or fitting and shall meet the applicable requirements of the specification under which the pipe or fitting is produced.

The surfaces of the bell and socket shall be smooth and free from holes, laps, and defects of every nature which unfit them for the use for which they are intended. The bell and socket shall be subject to suitable joint tests for leakage. Such tests shall be made at sufficiently frequent intervals to assure that the joints will be tight when properly assembled.

The annular recesses of the socket, the bolt circle, and the bell flange shall be concentric within the tolerances of Table 11.1. The dimensions of the bell and socket, the bolt circle, and the spacing of the bolt holes shall be gaged in accordance with Table 11.1, at sufficiently frequent intervals to assure dimensional control.

Sec. 11–6—Glands

The iron in the glands shall meet the requirements of Class 25 of Standard Specifications for Gray Iron Castings —ASA G25.1 (ASTM A48). The preferred acceptance test under this specification shall be the transverse test, although the tension test may be used for qualification. The minimum strengths under this specification are shown in the following table:

	1.2-in. Diam. Bar, 18-in. Supports, Load at Center	2-in. Diam. Bar, 24-in. Supports, Load at Center
Minimum transverse breaking loads	2,000 lb	6,800 lb
Minimum tensile strength	25,000 psi	

The annular surfaces of the gland lip and the bolt circle shall be concentric. Their accuracy within the tolerances of Table 11.1 shall be checked by gaging at sufficiently frequent intervals to assure dimensional control.

The surfaces of the gland shall be smooth and free from defects of every nature which would unfit them for the use intended.

Glands shall be coated with a bituminous dip or paint unless otherwise specified.

Sec. 11–7—Bolt Holes

Bolt holes for pipe and fittings may be drilled or cored. All bolt holes shall be free of sand or projections of iron that would interfere with the fit of the bolts. Drilled holes shall be

gaged as to location, at sufficiently frequent intervals to assure dimensional control. All cored holes shall be gaged for diameter of bolt circle, location and size of holes, and concentricity with the socket or gland lip.

Sec. 11–8—Gaskets

Manufacture. The gasket shall be made of a vulcanized crude-rubber compound and, unless otherwise specified, the rubber shall be first-grade plantation rubber. All surfaces of the gaskets shall be smooth except for the specified marking, and free from imperfections, and the gaskets shall be free from any porosity. Tips or backs, if specified for special conditions, shall be applied as shown in Fig. 11.2 and shall be well bonded to the rubber.

Service. Plain rubber gaskets are standard for joints in water service or other service where the gas or liquid being conveyed or the temperature is not deteriorating to the gaskets. Tips of duck, thiokol-impregnated duck, synthetic rubber of various kinds, lead, and other materials can be applied for special services. Duck or other reinforcing material also may be applied to the back of the gaskets. Gaskets other than plain rubber shall be specified on the purchase order.

Physical Properties and Tests. The following tests shall be made by the gasket manufacturer in accordance with Federal Specification ZZ–R–601a or the equivalent ASTM specification. The test specimen for all tests except hardness shall be die cut from routine production gaskets. It shall have a gage length of 2 in. with a width of 0.25 in. and a thickness of 0.10 in. The gaskets selected for testing shall be such as to represent the full range of quality of the lot from which the order is to be filled. For each 5,000 gaskets produced or for any week's production of less than 5,000 gaskets, at least one set of tests shall be made:

1. Ultimate tensile strength
2. Ultimate elongation
3. Elongation at 1,000-psi stress—limits: 110–190 per cent
4. Permanent set—the sample shall be clamped for 10 min at the elongation produced by 1,000-psi stress, relieved of load, and measured after 10 min rest; the permanent set, expressed as a percentage of the original gage length, shall not exceed 5 per cent
5. Hardness—hardness tests shall be made on each batch of production gaskets; this test shall be made with a Type A durometer; the preferred hardness is 70–75 durometer.

Sec. 11–9—Bolts and Nuts

High-strength, heat-treated cast iron tee-head bolts with hexagon nuts shall be the standard bolts; they shall be in accordance with Fig. 11.3 and meet the requirements hereinafter specified. If special types of bolts are required by the purchaser, agreement as to such shall be made at the time of placing the order.

Standard bolts and nuts shall be manufactured under close metallurgical control, and the finished bolt and nut assemblies shall be subject to proof testing with the load applied between the head and the nut in a suitable test machine. Proof tests shall be made at sufficiently frequent intervals to assure conformance with the proof test loads. The following proof test loads shall be applicable and shall not permanently stretch the bolt:

Size *in.*	Proof Test Load *lb*
⅝	10,000
¾	15,000
1	24,500
1¼	40,000

The bearing surface of the cast nut shall be smooth, and the nut shall be tapped at right angles to this surface to insure axial loading.

Bolts and nuts shall be coated with a rust-preventing lubricant after threading or tapping. They shall be packed in suitable containers, which shall be marked with the number of bolts, the size and type of bolts, and the net weight.

Sec. 11–10—Foundry Records

A record of the specified tests, including those of glands, bolts, and gaskets, shall be made and retained for at least one year. Upon request, such records will be available to the purchaser at the foundry. If written transcripts of the results of any tests are desired, this fact shall be noted in the order for the material.

Sec. 11–11—Inspection by Purchaser

(a) The purchaser shall have access at all times to those parts of the manufacturer's plant which concern the manufacture of articles being made for him. If the purchaser desires to inspect the manufacture of glands, bolts, and gaskets which may be made by subcontractors, special arrangements must be made therefor at the time of placing the order. He may inspect the joint material and may reject any of the material which is not in conformity with the specification.

(b) The manufacturer shall provide all tools, testing equipment, materials, labor, and facilities necessary for the required testing and inspection of the joint at the foundry.

(c) The purchaser shall make written report daily to the foundry office of all material rejected, with the causes for rejection.

Sec. 11–12—Inspection and Certification by Manufacturer

Should the purchaser have no inspector at the works, the manufacturer shall, if required by the purchaser, furnish a sworn statement that the inspection and all of the tests have been made and met as specified.

Notes on Method of Installation

These notes are not parts of the Standard, but are given for information only.

The successful operation of the mechanical joint specified requires that the spigot be centrally located in the bell and that adequate anchorage shall be provided where abrupt changes in direction and dead ends occur.

The rubber gasket seals most effectively (particularly when sealing gas) if the surfaces with which it comes in contact are brushed thoroughly with a wire brush just prior to assembly. This thorough brushing removes all loose rust or foreign material which may be present and provides clean surfaces which should be brushed with soapy water just prior to slipping the gasket over the spigot end

and into the bell. Soapy water brushed over the gasket prior to installation also removes loose dirt and lubricates the gasket as it is forced into its retaining space.

For water and gas service, the normal range of bolt torques to be applied to standard cast iron bolts in a joint are:

Size *in.*	Range of Torque *ft-lb*
⅝	40– 60
¾	60– 90
1	70–100
1¼	90–120

The above torque loads may be applied with torque measuring or indicating wrenches. Torque wrenches may be used to check the application of approximate torque loads applied by men trained to give an average pull on a definite length of regular socket wrench. The following lengths of wrenches should satisfactorily produce the above ranges of torques when used by the average man:

Size *in.*	Length of Wrench *in.*
⅝	8
¾	10
1	12
1¼	14

When tightening bolts, it is essential that the gland be brought up toward the pipe flange evenly, maintaining approximately the same distance between the gland and the face of the flange at all points around the socket. This may be done by partially tightening the bottom bolt first, then the top bolt, next the bolts at either side, and last, the remaining bolts. Repeat this cycle until all bolts are within the above range of torques. If effective sealing is not attained at the maximum torque indicated above, the joint should be disassembled and reassembled after thorough cleaning. Overstressing of bolts to compensate for poor installation practice is to be avoided.

TABLE 11.1—*Mechanical-Joint Dimensions, in Inches*

Size	A	B	C	D	F	θ	X	J	K_1		K_2	L	M	O	P	S		Bolts		
									Centrifugal Pipe	Pit Cast Pipe and Fittings						Centrifugal Pipe	Pit Cast Pipe and Fittings	No.	Size	Length
3	±.06 3.96	2.50	±.04 4.84	+.06 −.04 4.94	+.07 −.03 4.06	28°	+.06 −.0 ¾	±.06 6.19	−.06 7.62	−.12 7.69	−.12 7.69	−.06 .94	−.06 .62	.31	.63	−.05 .47	−.10 .52	4	⅝	3
4	±.06 4.80	2.50	±.04 5.92	+.06 −.04 6.02	+.07 −.03 4.90	28°	+.06 −.0 ¾	±.06 7.50	−.06 9.06	−.12 9.12	−.12 9.12	−.06 1.00	−.06 .75	.31	.75	−.05 .55	−.10 .65	4	⅝	3½
6	±.06 6.90	2.50	±.04 8.02	+.06 −.04 8.12	+.07 −.03 7.00	28°	+.06 −.0 ¾	±.06 9.50	−.06 11.06	−.12 11.12	−.12 11.12	−.06 1.06	−.06 .88	.31	.75	−.05 .60	−.10 .70	6	⅝	3½
8	±.06 9.05	2.50	±.04 10.17	+.06 −.04 10.27	+.07 −.03 9.15	28°	+.06 −.0 ¾	±.06 11.75	−.06 13.31	−.12 13.37	−.12 13.37	−.08 1.12	−.08 1.00	.31	.75	−.05 .66	−.12 .75	6	¾	4
10	±.06 11.10	2.50	+.06 −.04 12.22	+.06 −.04 12.34	+.07 −.03 11.20	28°	+.06 −.0 ¾	±.06 14.00	−.06 15.62	−.12 15.69	−.12 15.62	−.08 1.19	−.08 1.00	.31	.75	−.06 .72	−.12 .80	8	¾	4
12	±.06 13.20	2.50	+.06 −.04 14.32	+.06 −.04 14.44	+.07 −.03 13.30	28°	+.06 −.0 ¾	±.06 16.25	−.06 17.88	−.12 17.94	−.12 17.88	−.08 1.25	−.08 1.00	.31	.75	−.06 .79	−.12 .85	8	¾	4
14	+.05 −.08 15.30	3.50	+.07 −.05 16.40	+.07 −.05 16.54	+.06 −.07 15.44	28°	+.06 −.0 ¾	±.06 18.75	−.08 20.25	−.12 20.31	−.12 20.25	−.12 1.31	−.12 1.25	.31	.75	−.08 .85	−.12 .89	10	¾	4
16	+.05 −.08 17.40	3.50	+.07 −.05 18.50	+.07 −.05 18.64	+.06 −.07 17.54	28°	+.06 −.0 ¾	±.06 21.00	−.08 22.50	−.12 22.56	−.12 22.50	−.12 1.38	−.12 1.31	.31	.75	−.08 .91	−.12 .97	12	¾	4½
18	+.05 −.08 19.50	3.50	+.07 −.05 20.60	+.07 −.05 20.74	+.06 −.07 19.64	28°	+.06 −.0 ¾	±.06 23.25	−.08 24.75	−.15 24.83	−.15 24.75	−.12 1.44	−.12 1.38	.31	.75	−.08 .97	−.15 1.05	12	¾	4½
20	+.05 −.08 21.60	3.50	+.07 −.05 22.70	+.07 −.05 22.84	+.06 −.07 21.74	28°	+.06 −.0 ¾	±.06 25.50	−.08 27.00	−.15 27.08	−.15 27.00	−.12 1.50	−.12 1.44	.31	.75	−.08 1.03	−.15 1.12	14	¾	4½
24	+.05 −.08 25.80	3.50	+.07 −.05 26.90	+.07 −.05 27.04	+.06 −.07 25.94	28°	+.06 −.0 ¾	±.06 30.00	−.08 31.50	−.15 31.58	−.15 31.50	−.12 1.62	−.12 1.56	.31	.75	−.08 1.08	−.15 1.22	16	¾	5
30*	32.00	4.00	33.29	33.46	32.17	20°	1¼	36.88	39.12	39.12	39.12	1.81	2.00	.38	1.00	1.20	1.50	20	1	5½
36*	38.30	4.00	39.59	39.76	38.47	20°	1¼	43.75	46.00	46.00	46.00	2.00	2.00	.38	1.00	1.35	1.80	24	1	5½
42*	44.50	4.00	45.79	45.96	44.67	20°	1¼	50.62	53.12	53.12	53.12	2.00	2.00	.38	1.00	1.48	1.95	28	1¼	6
48*	50.80	4.00	52.09	52.26	50.97	20°	1¼	57.50	60.00	60.00	60.00	2.00	2.00	.38	1.00	1.61	2.20	32	1¼	6

* Dimensions are tentative for these sizes.

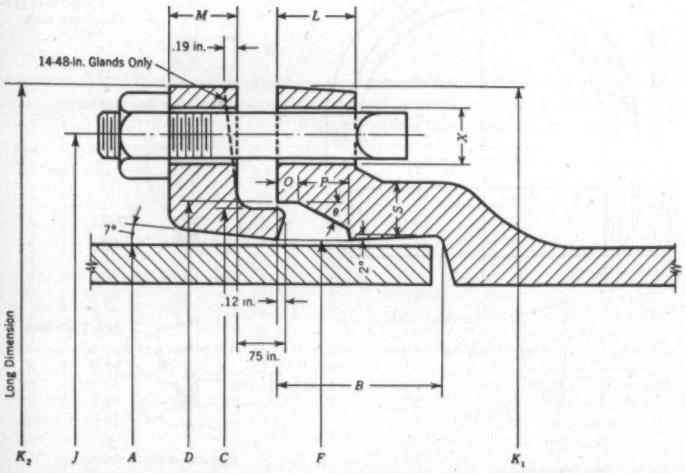

Fig. 11.1. 3–48-in. Standard Mechanical-Joint Dimensions (See Table 11.1)

Notes

1. The thickness of the bell, *S*, shall in all instances be equal to, and generally exceeds by at least 10 per cent, the nominal wall thickness of the pipe or fitting of which it is a part.

2. Cored holes may be tapered an additional 0.06 in. in diameter.

3. In the event of ovalness of the spigot OD, the mean diameter measured by a circumferential tape shall not be less than the minimum diameter shown in the table. The minor axis shall not be less than the above minimum diameter plus an additional minus tolerance of 0.04 in. for sizes 8–12 in., and 0.07 in. for sizes 14–24 in.

4. K_2 is OD of glands across bolt holes; outside of gland may be polygon shape.

5. Dimensions of 30–48-in. sizes are tentative.

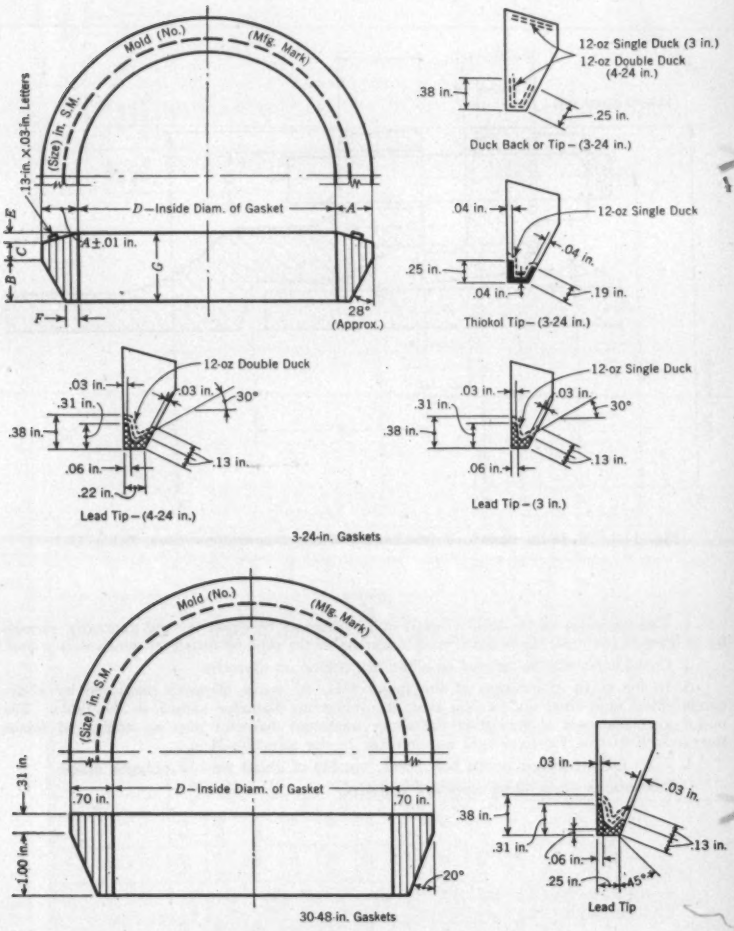

Fig. 11.2. Mechanical-Joint Gaskets (See Table 11.2 and Notes)

TABLE 11.2

3–48-in. Mechanical-Joint Gasket Dimensions, in Inches

Pipe Size	Pipe OD	Dimensions of Plain Rubber Gaskets						
		A ±.01 in.	*B*	*C*	*D* +0% −1%	*E*	*F* ±.01 in.	*G* ±.02 in.
3	3.96	.48	.62	.31	3.90	.12	.15	1.05
4	4.80	.62	.75	.31	4.73	.16	.22	1.22
6	6.90	.62	.75	.31	6.80	.16	.22	1.22
8	9.05	.62	.75	.31	8.91	.16	.22	1.22
10	11.10	.62	.75	.31	10.93	.16	.22	1.22
12	13.20	.62	.75	.31	13.00	.16	.22	1.22
14	15.30	.62	.75	.31	15.07	.16	.22	1.22
16	17.40	.62	.75	.31	17.14	.16	.22	1.22
18	19.50	.62	.75	.31	19.21	.16	.22	1.22
20	21.60	.62	.75	.31	21.28	.16	.22	1.22
24	25.80	.62	.75	.31	25.41	.16	.22	1.22
30*	32.00				31.50			
36*	38.30				37.62			
42*	44.50				43.67			
48*	50.80				49.80			

* Dimensions are tentative for these sizes.

Notes

1. Duck backing can be omitted from lead-tipped gaskets provided sufficient bonding of the lead to the rubber is obtained.
2. Tipped or backed gaskets may be made in the same mold as plain rubber gaskets, but the inside diameter of such reinforced portions shall not exceed the "Pipe OD."
3. Dimensions of 30–48-in. sizes are tentative.

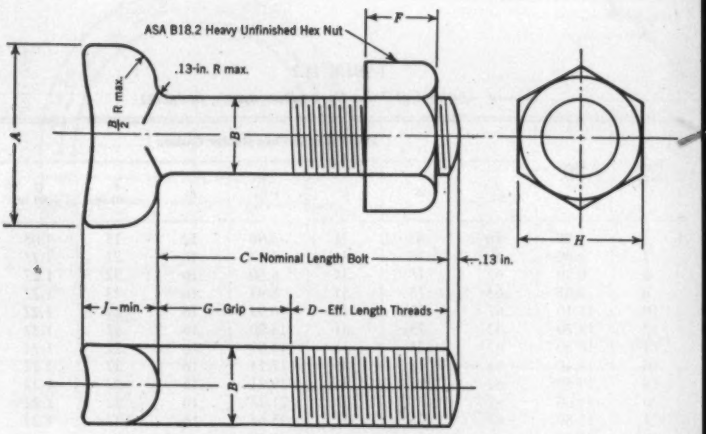

Fig. 11.3. Mechanical-Joint Bolts and Nuts

TABLE 11.3

Mechanical-Joint Bolt and Nut Dimensions, in Inches

Nom. Size	*A* ±.12	*B* ±.03	*C* ±.12	*D* +unlim. −.06	*E**	*F* +.12 −.00	*G*	*H*†	*J*	Weight of 100 Bolts With Nuts *lb*
⅜×3	1.50	.625	3.00	1.50	11	.625	1.50	1.062	.625	47
⅜×3½	1.75	.75	3.50	2.00	10	.75	1.50	1.25	.75	77
¾×4	1.75	.75	4.00	2.00	10	.75	2.00	1.25	.75	82
¾×4½	1.75	.75	4.50	2.00	10	.75	2.50	1.25	.75	87
¾×5	1.75	.75	5.00	2.00	10	.75	3.00	1.25	.75	93
1 ×5½	2.25	1.00	5.50	2.50	8	1.00	3.00	1.625	1.00	192
1¼×6	2.50	1.25	6.00	2.50	7	1.25	3.50	2.00	1.25	334

*Standard threads per inch, coarse-thread series, Class 2 fittings (ASA B1.1).
† Tolerance—for ⅜-in.: +0.0, −0.04; ¾-in.: +0.0, −0.06; 1-in.: +0.0, −0.08; 1¼-in.: +0.0, −0.10 in.

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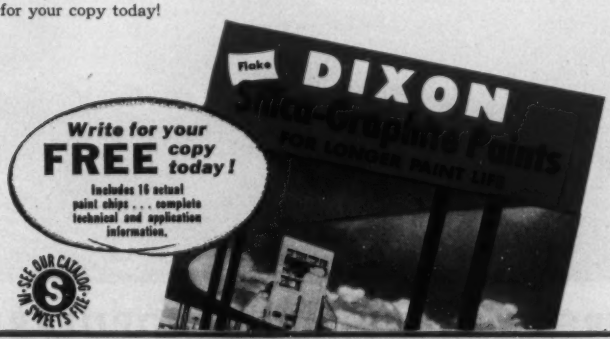
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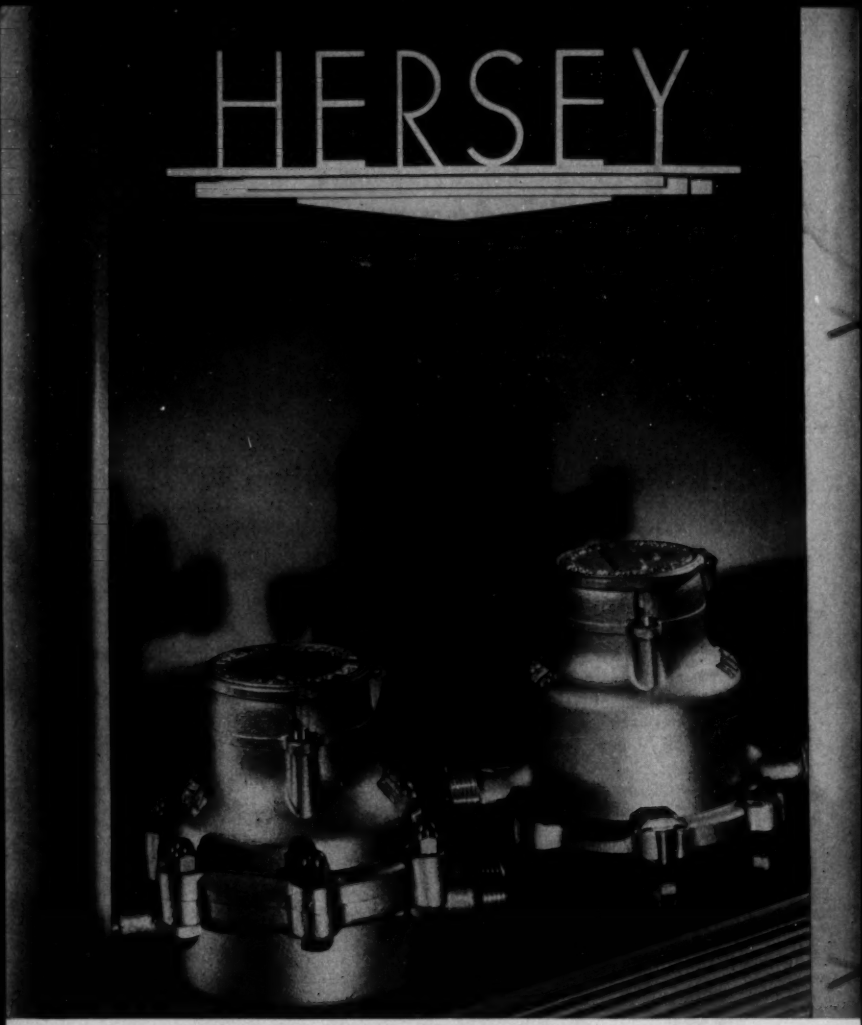

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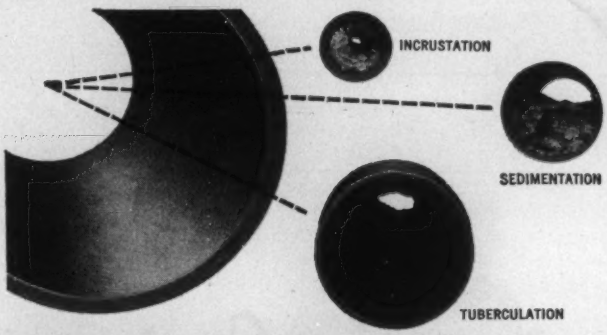

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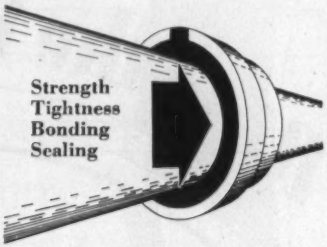

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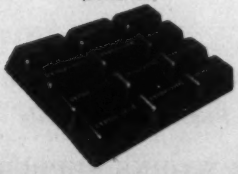

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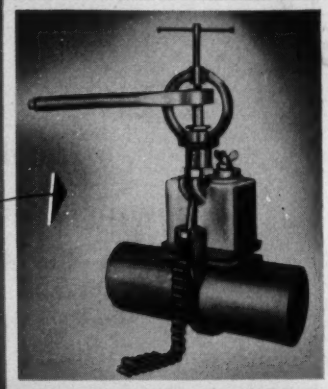

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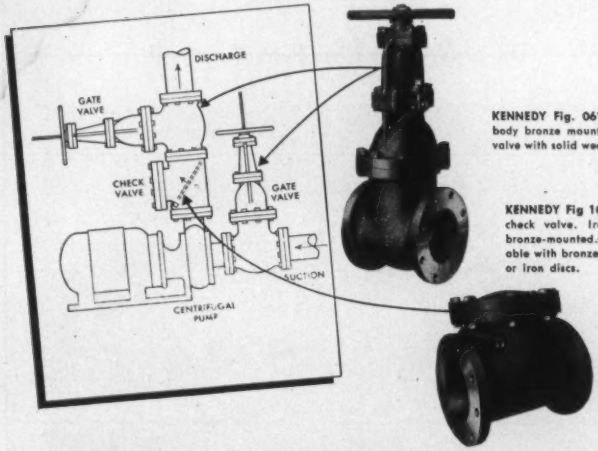

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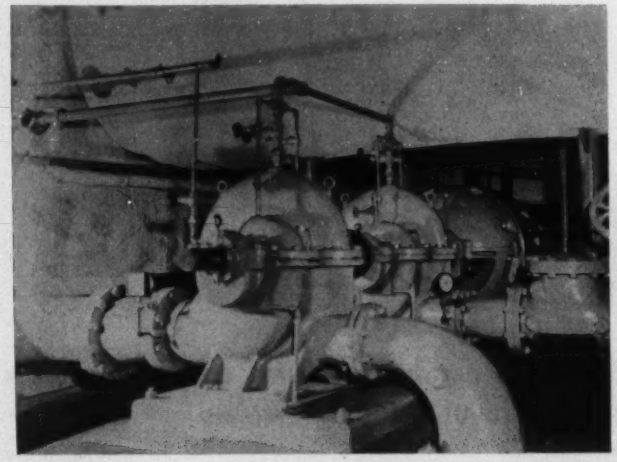

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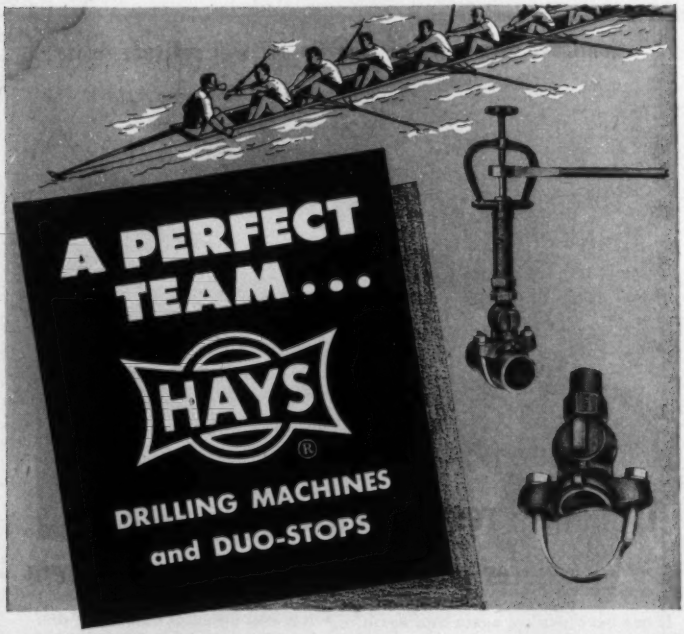

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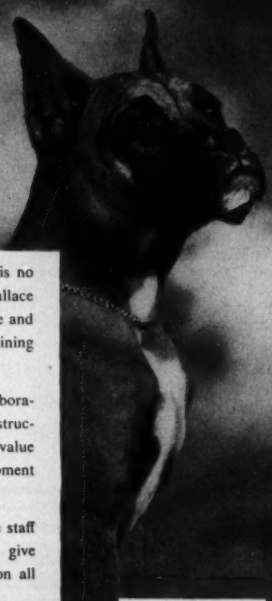

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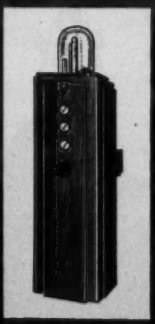

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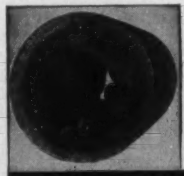

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Combination automatic control both directions through the valve.

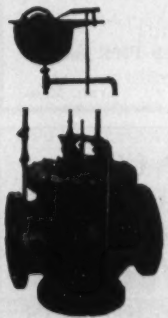
Maintains desired discharge pressure regardless of change in rate of flow

REDUCING VALVE

Regulates pressures in gravity and pump systems; between reservoirs and zones of different pressures, etc.

Maintains safe operating pressures for conduits, distribution and pump discharge

SURGE-RELIEF VALVE



Maintains levels in tank, reservoir or basin

1. As direct acting

2. Pilot operated and with float traveling between two stops, for upper and lower limit of water elevation.

FLOAT VALVE

Electric remote control—solenoid or motor can be furnished

REMOTE CONTROL VALVE

Adapted for use as primary or secondary control on any of the hydraulically controlled or operated valves.

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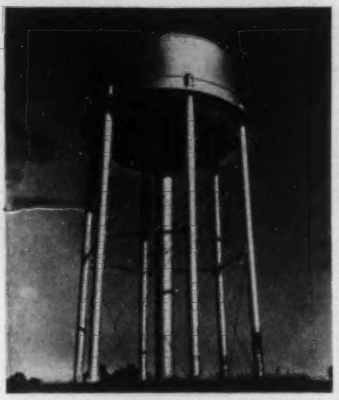

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CEMENT LINED SERVICE PIPE:—
Resists Corrosion
Has proven practical for three-quarters of a century
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The Old Port Washington, Wisconsin, Water plant is now one of the most modern filter plants in the country.

The illustration shows the pump room, entirely equipped with Rensselaer gate and check valves. All gate valves 4 inch and larger in the entire plant are also Rensselaer.

Rensselaer products are well above specifications not only in general characteristics but in the thickness of metal at important points. They are designed with the "know how" which goes back to the early days of water systems, and they are built by technicians many of whom have been with the Company 35 years or more.

You don't have to build a new plant in order to modernize with Rensselaer. Ask our local representative why a replacement of a troublesome check valve will eliminate slam—why Rensselaer tapping sleeves save time, and why Rensselaer hydrants and valves will save maintenance on your service extensions.

Rensselaer VALVE COMPANY, TROY, N. Y.

GATE VALVES • FIRE HYDRANTS • SQUARE BOTTOM VALVES • CHECK VALVES • AIR RELEASE VALVES

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Sales representatives in principal cities

104

72nd

Annual Convention

NEW ENGLAND
WATER WORKS ASSOCIATION

September 28, 29, 30, 1953

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Poland Spring, Maine

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96%

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STILL IN SERVICE

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A detailed report of the survey has recently been published by the American Water Works Association. We have reprinted, by permission, the facts pertaining to cast iron water mains in a brochure which will be sent on request. Write to Cast Iron Research Association, Thomas F. Wolfe, Managing Director, 122 South Michigan Ave., Chicago 3, Ill.

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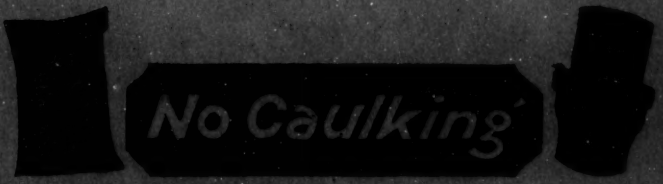
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